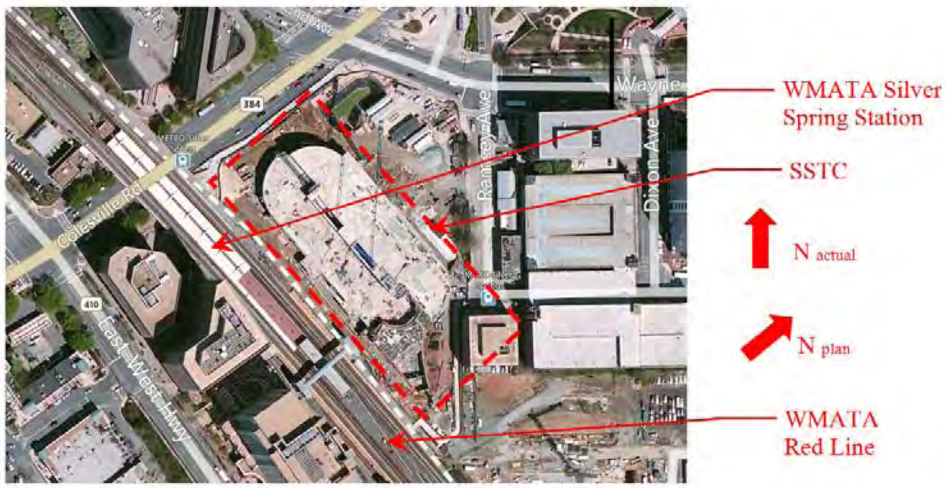
SILVER SPRING TRANSIT CENTER

Silver Spring, Maryland



Courtesy of Bing Maps

Silver Spring Transit Center Structural Evaluation of Superstructure

March 15, 2013

MONTGOMERY COUNTY 17 76 MARYLAND

Prepared for:
Montgomery County
101 Monroe Street, Third Floor
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March 15, 2013

Office of the Montgomery County Attorney
Division of Finance and Procurement
101 Monroe Street, 3rd Floor
Rockville, Maryland 20850

Attn: John Markovs
Deputy County Attorney

RE: Silver Spring Transit Center Job No. 2012-13
Above-Grade In Situ Structural (Project) Evaluation

Gentlemen:

I. SCOPE

In accordance with Montgomery County, Maryland's (Montgomery County) request and pursuant to KCE Structural Engineer's (KCE) agreement with Montgomery County, we have completed our evaluation of the in situ conditions of the structural frame of the Silver Spring Transit Center (SSTC) as described herein. Our review was based on the documents provided, our field investigation observations, and our engineering analyses.

We reserve the right to amend/modify this report when and if new or additional information is provided to us.

We have no direct knowledge of and offer no warranty regarding the condition of concealed construction or subsurface conditions beyond what was found in our evaluation. Any comments we offer regarding concealed construction are our professional opinions based on analyses, in situ testing, and our joint engineering experience and judgment, and are derived in accordance with the standard of care and practice for evaluations of building structures.

Comments in this report are intended to be representative of observed and tested conditions.

Conceptual repair recommendations discussed herein will require additional engineering design and document preparation prior to implementation.
We have made every effort to reasonably study the various areas of concern, those identified during our site visits, and those noted in our review of documentation and our analyses. If there are perceived omissions or misstatements in this report regarding the observations made, we ask that they be brought to our attention as soon as possible so that we have the opportunity to address them fully and in a timely manner.

This report summarizes our document review, field investigation, and engineering analyses results.

This report is based on our review of the information provided to us as listed in Attachment 1, including, but not limited to:

- Tourney Consulting Group LLC report dated March 14, 2012
- CTL Group, LLC reports dated March 2, 2012
- Adojam Report (undated)
- Desman Reports
- Greenhorn and O'Mara November 2011 survey
- Pennoni November 2011 reports
- Survey details from Facchina dated August 2011
- Concrete compressive break values
- Approved VSL post-tensioning shop drawings
- Approved Gerdau Ameristeel mild steel shop drawings
- Robert B. Balter Company (RBB) field inspection and compressive strength testing reports
- Limited Project correspondence
- Stressing records
- VSL proposal for Pour Strip remediation
- Memorandum of Understanding (MOU) between Montgomery County and the Washington Metropolitan Area Transit Authority (WMATA)
- Parsons Brinckerhoff (PB) calculations of slabs and beams
- PB calculations dated 2/27/12 and 5/6/12

Further, we have reviewed correspondence dated March 2012, March 14, 2012, and June 21, 2012 from Simpson Gumpertz Heger (SGH), as well as surveys by WMATA, Facchina Construction Company, Inc. (Facchina), and the subcontractors and consultants.

We also reviewed letters regarding the status of the building structure prepared after the issuance of the SGH correspondence by PB, one dated February 27, 2012 and two others dated May 22, 2012 (one in response to the SGH report and one analyzing the structure based on "thin slabs").

The Contracts we reviewed included those between:

- Montgomery County and Foulger Pratt Construction (FP)
- Montgomery County and The Robert B. Balter Company (RBB) for inspection and materials testing services and as the Montgomery County Special Inspections Program Special Inspector
- Montgomery County and Parsons Brinckerhoff Quade and Douglas, Inc. (PB) (aka Parsons Brinckerhoff Americas, Parsons Brinckerhoff, Inc.) (PB) for Project design

- Montgomery County and Parsons Brinckerhoff Quade and Douglas, Inc. (PB) (aka Parsons Brinckerhoff Americas, Parsons Brinckerhoff, Inc.) (PB) for on-site “Construction Project Management”
- Facchina and FP
- Facchina and R&R Reinforcing, Inc. (the installer of mild-steel and post-tensioning reinforcing
- (Purchase Order) Facchina and VSTRUCTURAL LLC (VSL) (the post-tensioning supplier).

Our initial field testing and evaluation was based on review of randomly selected representative areas and concrete cores.

After our initial assessments, we determined that additional field investigations were needed to verify in situ concrete strengths, air content, and water/cement ratios, as well as the need to perform two-dimensional and three-dimensional surveys of the structure, pulse echo testing of slab cracks, determining post-tensioning clearances, reinforcing clearances, number and locations of post-tensioning ducts and mild reinforcing steel. In addition, we performed chain drags and Impulse Response testing to locate possible voids in the slabs and Impact-Echo testing to determine crack widths and depths on the top surface of framed decks (not under “pedestrian areas”).

We also performed structural analyses of the structure considering both the as-designed and as-built conditions.

We have also been asked to develop our professional engineering opinion as to:

1. The ability of the building to support the loads it was to have been designed to support
2. The durability and maintenance of the as-built structure

   and
3. The causes of concerns raised previously by others and items we found in our review.

Finally, we were asked to provide a concept for the repairs/remediation necessary. We will separately develop repair and remediation documents at your request to resolve the strength, durability, fire resistance, and maintenance issues we have found.

II. EXECUTIVE SUMMARY

The SSTC is a post-tensioned concrete transit facility located adjacent to a WMATA passenger rail station in downtown Silver Spring, Maryland, located at the intersection of Colesville Road and Wayne Avenue. The SSTC facility provides three levels of access for vehicular traffic, commonly referred to as Levels 300, 330, and 350. We understand the 300 and 330 levels are to serve as a terminal for local and regional bus service, and the 350 (third level) is to provide “kiss and ride” passenger drop-off and pick-up for private vehicles and taxis. The site topography (slope) allows direct vehicular access to each of the three levels and pedestrian access to the rail station. Pedestrian access between the three levels is provided by stairs, escalators, ramps, and elevators located near the center of the facility. The SSTC is primarily open to the elements, with an enclosed office suite/commuter store at its Eastern end.

As construction progressed, concerns arose regarding the concrete decks when a post-tensioning cable popped out of the concrete and cracking was observed.

These concerns included but were not limited to:

1. The thickness of the concrete slabs;
   and as noted
2. Visible evidence of extensive cracking in the slabs;
3. Exposed post-tensioned ducts.

Montgomery County retained KCE on June 18, 2012 (Notice to Proceed issued on June 20, 2012), to conduct an extensive document review and structural evaluation of the SSTC structure. To assist with their evaluation, KCE retained Wiss Janney Elstner Associates, Inc. (WJE) and Walter P Moore and Associates, Inc. (WPM), who in turn retained other consultants/subcontractors. Information about the Project team is included in Attachment 2.

Based on our document review, field investigation, and engineering analyses, the SSTC will require strengthening and repairs to meet Building Code and WMATA requirements. The structure, however, can continue to safely support current construction-phase loading. Specific issues identified during our assessment are described herein.

A. Applicable Code Requirements

The SSTC, based on the building permit issued, must comply with the 2003 edition of the International Building Code (IBC 2003) and its referenced documents, such as, but not limited to, ACI 318-02, Codes and publications referenced therein, and Industry Standards. In addition, the SSTC was to comply with the WMATA Manual of Design Criteria and WMATA Standards, which were to have been incorporated into the design, which have requirements that are, in some cases, more stringent than those prescribed by IBC 2003. For example, IBC 2003, ACI 318-02, and the WMATA Manual of Design Criteria and WMATA Standards have requirements, some more restrictive than others, to minimize cracking by limiting tensile stresses by design in the concrete under service conditions.

B. Design Review and Analysis

Our review of the Contract Documents, Requests for Information (RFIs) and their responses, Architectural Supplemental Instructions (ASIs), and numerous sketches and field changes indicated, among other things, lack of coordination during design between elements, such as:

- Electrical and other embedded items interfering with reinforcing and post-tensioning
- Mild reinforcing interfering with post-tensioning
- Post-tensioning interfering with mild reinforcing
- Post-tensioning stressing pockets in concrete conflicting with mild reinforcing
- Slab geometry and sloping to drains vis-à-vis specified slab thickness

The design also:

- Induced forces that "overbalanced" the structure due to post-tensioning forces that exceeded the actual weight of the slabs, beams, and girders, inducing cracks of the structure.
- Did not take into account various required limitations on stress induced during initial post-tensioning. Those stresses also induced cracking into the building during the construction work effort.

- Did not accommodate the stress caused by "restraint" forces due to the as-designed integral concrete walls, columns, and girders, which induced cracking in the slabs and in those elements themselves.
- Did not incorporate into the Contract Documents, the required WMATA Manual of Design Criteria, and the WMATA Standards:
    - Required quantities of corrosion inhibitors in the concrete utilized in the structure
    - Maximum expansion joint spacing
    - Maximum allowed extreme fiber stress
    - Testing requirements
    - Curing requirements
        - Temperature
        - Time
    - Pouring limitations
    - Slump limitations
- Underdesigned certain elements of the structure to resist shear forces and torsion forces.
- Did not accommodate the County roadway depression necessary for the Bonifant (Ramsey) Street turnaround.
- Did not include delineation of post-tensioning requirements in Pour Strips at Level 330.
- Did not properly accommodate the fire rating requirements of structural elements as required by the applicable IBC 2003 Code.
- Did not issue revised drawings for updated permits indicating the changes made in the Conformed Set or during construction.

C. <u>Inspections</u>

The inspection efforts performed during construction on the Project were not per the Contract Document requirements, WMATA requirements, Statement of Special Inspections, or their Contract, which may have yielded incorrect in situ materials strength information (cylinders) used for stripping and stressing decisions and strength evaluations.

D. <u>Concrete Material Properties</u>

Based on in situ sampling and testing performed, the concrete in the structural decks has lower compressive strength than required by the Contract Documents. The compressive strength is also lower than that reported by construction period sampling and testing. There are several possible explanations for the discrepancy between the construction period concrete test results and the in situ core test results developed during our current assessment, as discussed further in this report.

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E. Slabs

1. Pour Strip Slabs

The East and West Pour Strips on Level 330 were constructed without post-tensioning. In addition, the Pour Strip at the West end of the facility at Level 330 was constructed without reinforcing steel to control temperature and shrinkage forces. The Pour Strip slabs, as designed, have sufficient capacity to support the required loads, but do not as built.

2. Typical Slabs

We visually observed representative slabs and performed a vertical elevation study. Slab thickness for the elevated slabs was to be 10 inches minimum based on the Construction Documents. With tolerances allowed by the Contract Documents, the minimum thickness of the slabs that would be in conformance with the Contract Documents would be 9-3/4 inches. The upper limit on slab thickness is 10-3/8", but that limit is brought into question because of the sloping of the top of the slab. Based on our three-dimensional measurements, variations in the in situ slab thickness result in approximately 21% of Level 330 and 22% of Level 350 on the "thin side" that do not comply with the Contract Document requirements. Our structural analysis indicates only slabs with thickness below 8-1/2 inches to 8-3/4 inches with 8,000 psi concrete and 9 inches based on the 6,970 psi calculated in situ concrete strength (as described herein) affects load-carrying capacity in limited areas.

Visual observations and nondestructive test results identified widespread cracking, thin cementitious patches on top of portions of the slabs, exposed post-tensioning tendons at the top surface of the slabs and reinforcing steel at the top of the elevated slabs with less than the specified concrete cover (after accounting for allowed tolerances). The observed cracking appears to be related to design issues, restraint conditions, concrete placement, and the curing processes.

Our analysis of randomly selected representative areas of the as-designed and as-built structure indicates that the slabs have adequate capacity to support the Code and WMATA required design loads. However, as-designed, the required stress limitations to control cracking at initial and service conditions were exceeded.

Our analysis of the as-built post-tensioned slabs indicates slab areas with thicknesses below approximately 9 inches and with compressive strengths at or below 6,970 psi do not have adequate shear capacity in certain locations to support the design loads (the areas less than 9 inches thickness are limited in extent and therefore do not limit overall load-carrying capacity). In addition, the as-designed analysis indicates the initial and service level stresses were exceeded.

In summary, structural slabs in the SSTC facility have cracked as a result of a combination of design errors and omissions, insufficient design/construction coordination, and the as-built concrete material properties as placed. In addition, the restraint of the post-tensioned slab system, was caused by pouring slabs, as designed,

without bond breakers between the slab elements and the stiff concrete walls, pouring columns integral with those walls (which are supporting the stiff girders) and not dealing with those stresses and forces in design cause additional cracking.

F. Beams

1. Pour Strip Beams

Beams as-designed at the Pour Strip periphery and interior have adequate strength to support the Code and WMATA required design loads with the specified concrete strength of 8,000 psi and the calculated in-place strength of 6,970 psi.

2. Post-Tensioned Beams

We visually observed and surveyed randomly selected representative post-tensioned beams in the SSTC facility. Observations related to the beams include post-tensioning tendon profile deviations and concrete cracks. The cracks were predominantly located at the beam ends.

Our analysis determined that certain beams in the drive lanes do not have adequate strength to support the design loads for both as-designed and as-built when the in-situ concrete strength is less than the specified concrete strength, i.e., 8,000 psi vs. 6,970 psi (the in-place strength as described herein). In addition, we again determined that initial stress limits required by Code limitations of induced stresses to control cracking at initial and service conditions were exceeded.

G. Girders

We visually observed and surveyed randomly selected representative girders in the SSTC facility. Observations related to the girders include localized post-tensioned tendon locations and elongation deviations and concrete cracks. The cracks were typically located throughout the length of the girders.

Our analysis determined that the girders at limited locations do not have adequate strength to support the design loads in combined shear and torsion with either 8,000 psi or 6,970 psi concrete. In addition, stress limits at initial and service conditions exceeded design limits.

H. Columns

We visually observed representative columns. The columns evidence cracks in the exterior faces of the columns, relocated reinforcement (as directed), and insufficient concrete cover of the column reinforcement.

Our analysis indicates that the columns have adequate strength to support the Code and WMATA required design loads.

I. Durability Analysis

We conducted a durability analysis of the as-built SSTC facility. The WMATA Manual of Design Criteria requires a structure to have a minimum 50-year service life.

Our durability analysis suggests the following:

1. The location and depth of top surface slab cracks leave the structure vulnerable to water and chloride-ion intrusion, which reduces the time to initiation of corrosion to immediately after the first winter application of deicing salts at locations where epoxy coating is not present or is compromised.

2. Water infiltrating into the cracks will freeze and thaw resulting, over time, in widening and lengthening of the existing cracks.

3. Concrete cover observed and measured over mild reinforcement is less than required by Contract Documents and therefore reduces time to corrosion initiation.

4. Locations with exposed or near-surface post-tensioning tendon ducts at the top surface of the slab are vulnerable to mechanical damage, which would lead to rapid initiation of tendon corrosion.

5. Service life modeling (Stadium©) for non-cracked areas of the concrete slabs suggests that time to initiation of corrosion can be as early as 12.5 years for areas with concrete cover that is 3/4 inch, or less where epoxy coating is compromised.

6. The durability of concrete columns with concrete cover less than 2 inches is compromised.

7. Beam elements that have displayed cracking are also vulnerable to chloride intrusion, carbonation, increased acidity in the concrete due to exposure, and freeze-thaw damage, thus compromising the durability of those portions of the structure.

8. In situ entrained air content of the concrete does not meet the requirements of the Contract Documents, making the concrete vulnerable to freeze-thaw damage during its 50-year design life.

   We would note the specified entrained air in the concrete generally conforms with Industry Standards for this geographic location. However, the in situ air void spacing (not a Contract Document limitation) is not ideal to resist freeze-thaw.

9. There is a large quantity of entrapped air (introduced during mixing and finishing) that also limits the freeze-thaw resistance.

J. Fire Rating

We performed a limited review of Code requirements for fire rating of the structural elements of the SSTC structure. Two- and three-hour fire ratings are required for various components for

a Type IA building, the classification noted on the Contract Documents and on the Building Permit documents.

The November 16, 2007 (various dates on the drawings) Permit drawings require two- and three-hour fire ratings for various structural elements, while the table on the Architectural drawings incorrectly indicates one-hour ratings are to be provided.

Based on our findings of nondestructive evaluations, the slabs, beams, and girders generally meet Code prescribed requirements for the two-hour fire rating, but not the three-hour rating. However, columns in the SSTC facility do not meet Code prescribed requirements for either two-hour or three-hour fire ratings due to insufficient cover.

K. Summary

In summary, the concrete in deck Pours 1A, 1B, 1E, 1H, and 2C has unacceptable concrete strength based on the ACI 318-02 requirements. The Pour Strips, due to their in situ conditions, are unacceptable.

The in-place concrete strength, when analyzed per ACI 214.4R-10, a statistical evaluation based on the strength of the secondary cores (78) extracted, is 6,970 psi.

L. Conceptual Recommendations

Based on our evaluation of the above-ground structural frame of SSTC, it is our opinion that remedial actions are required:

1. To provide the required strength of certain structural elements.
2. To provide long term durability of the decks and columns.
3. To achieve the required fire rating of certain columns.

Our conceptual recommendations follow:

1. Remove and replace existing Pour Strip slabs on Level 330 with appropriately designed and detailed Pour Strips before the overlay noted below is installed.

2. Increase the combined shear and torsional capacity of selected post-tensioned beams on Levels 330 and 350.

3. Enlarge certain columns to provide the required fire rating and increase durability.

4. Increase the combined shear and torsional capacity of selected post-tensioned girders to provide the required shear and torsion capacities.

5. Provide a properly detailed concrete overlay on the top surface for the slabs of Levels 330 and 350 in order to provide the required long-term durability.

   There are two approaches that can be adopted to address these slab concerns:

a. Design an unbonded overlay system including an appropriately designed wearing course for traffic loads and a properly detailed interstitial waterproofing layer.

or

b. Design a bonded topping slab.

6. The Owner should request a Code modification to change the building classification to Type IIA, and thereby the fire resistance requirements of the structure would be met by the existing conditions, other than columns with clear cover less than 2 inches minus tolerances.

Note that there may be a need for additional strengthening of the structure to accommodate the dead loads that exceed 35 psf in the drive aisles accommodated in the original design as noted on the Contract Documents.

All parties must understand there is a need for normal ongoing maintenance required for an exposed structure of this type to achieve its intended service life. That is to say, in order to achieve the intended service life, routine and periodic maintenance, including, but not limited to, maintaining expansion joints and sealants, epoxying cracks, and performing concrete repairs will be required. In addition, the application of deicing salts should be monitored during the winter months and washed off as soon as practical.

III. INTRODUCTION

The SSTC is a cast-in-place (CIP), post-tensioned reinforced concrete structure with two elevated structural levels and an at-grade level. The facility is located at 8400 Colesville Road, Silver Spring, Maryland.

During construction, circa 2010, concerns were raised about the thickness of the elevated slabs and positioning of the post-tensioning tendons.

In 2012, Montgomery County retained KCE, who in turn retained Wiss, Janney, Elstner Associates, Inc. (WJE), Walter P Moore and Associates, Inc. (WPM) and others to assess, as a team, the as-designed and as-built structure. The purpose of this report is to summarize our evaluations.

KCE is a full service structural engineering firm located in Washington, DC. KCE was founded over 45 years ago by Allyn Kilsheimer, PE and has over 30 full-time employees. KCE has designed and repaired over a thousand buildings across the United States and overseas, and has unique experience with collapse investigation and emergency assistance stabilization engineering. “High profile” projects include The Phoenix Project at the Pentagon; the restoration of the historic renovation of the Renwick Gallery; Blair House; Lafayette Square; and structural design of Federal Triangle; the United States Patent and Trademark Office; the Discovery Communications headquarters, Bethesda Metro Center, Metro Center, and numerous District of Columbia and federal government agency headquarters. KCE’s scope in the joint team effort on this Project was to perform extensive document review, to perform an

assessment of the original design and construction process, and to develop, in conjunction with WPM and WJE, conceptual repair recommendations.

WJE is a firm of structural engineers, architects, and materials scientists specializing in the assessment and repair of existing structures including bridges and buildings. WJE has over 500 employees and 19 offices across the United States. WJE performed a limited document review, review of KCE's as-designed analysis, extensive field investigations, an assessment of the concrete materials testing and an analysis of the as-built structure, and the statistical evaluation of concrete strength.

WPM is a firm of civil, structural, and forensic engineers specializing in the design of new buildings and assessment and repair of existing structures. WPM has over 300 employees and 13 offices across the United States. WPM provided limited nondestructive testing, Service Life Modeling, and engineering analysis of the structure for durability. In addition, WPM performed work product peer reviews of structural analyses by KCE and WJE throughout the course of our work.

Our team initially also included:

- American Petrographic Services, Inc. (APS) – concrete and grout materials testing
- RJ Lee Group (RJ Lee) – concrete material testing to calibrate the Service Life Model
- Rice Associates, Inc. (Rice) – three-dimensional scans and survey of the elevated slabs

We then added:

- The Erlin Company (TEC) – concrete materials testing
- Universal Construction Testing, Limited (UCT) – concrete materials testing
- Janney Technical Center (JTC) (WJE) – concrete materials testing and statistical evaluation (note: enclosed JTC petrographic and compressive reports are on WJE stationery)

Support contractors in the field were:

- Freyssinet USA – labor for evaluation openings and repair to openings
- Testing Technologies, Inc. – radiography of slabs
- Penhall Company – coring and extracting concrete cores
- United Rentals – scissor and aerial lifts

IV. BACKGROUND INFORMATION

The SSTC structure is primarily composed of post-tensioned, CIP concrete slabs, beams, and girders, supported by conventionally reinforced concrete columns. The concrete columns are typically supported on drilled shafts (aka caissons) bearing on rock. The shape of the building resembles an ellipse with a planar center area and curvilinear ends. Two-story tall concrete retaining walls located on the North and East sides of the structure conform the structure to the sloping site. Near the center of the structure, stairs, elevators, and escalators provide vertical access between Levels 305, 330, and 350. Attachment 3 illustrates a plan and photograph of each level.

The building overall plan dimensions are approximately 550 feet (Northwest-Southeast) by approximately 210 feet (Northeast-Southwest). (Note that plan North referenced herein is as per Contract Document indications).

The slab on ground is Level 305, the first elevated slab (2nd floor) is Level 330, and the second elevated slab is Level 350 (3rd floor). We have been advised, and based on our review of the Contract Documents, Levels 305 and 330 are intended to carry bus traffic while Level 350 is to function as a “kiss-and-ride” area. However, Level 350 appears to have also been designed to support bus traffic.

Drainage of the slabs was accommodated by design through grading (aka “cross-slope,” “camber,” “crowning”) of the tops of the slabs, directing water towards floor drains and catch basins.

Most of the concrete surfaces are exposed to direct weather (all of Level 350 and approximately half of Level 330) and the balance are exposed to wind-blown elements, with all levels subject to thermal changes. Consequently, virtually all of the structural elements are subjected to moisture changes and seasonal temperature variations including cyclic freezing and thawing, the application of de-icing salts in winter months, and normal ongoing concrete carbonation.

The structural Contract Document drawings stamped “Conformed Set Contractor to verify accuracy” (Attachment 4 first sheet only) and select structural Contract Document Specifications 03300 and 03381 noted RFP 7504510123 (Attachment 4A) both were issued on January 7, 2008. The Architectural Contract Document drawings, also dated January 7, 2008, are noted “RFP 75045 10123” on the cover sheet, but noted as “Conformed Set” on the structural drawings for the balance of the architectural set. We did not locate a “For Construction” issue set of drawings but understand the Conformed Set served in this capacity. The Conformed Set included Addenda 2, 3, and 4 of various dates and sketches SK9-25.

SSTC construction started in 2009 (based on a building permit issued 8/20/2009) (Attachment 5) and a set of permit drawings dated November 16±, 2007.

The Architect of Record (AOR) was/is Zimmer Gunsul Frasca Architects LLP, the Structural Engineer of Record (SEOR) was/is Parsons Brinckerhoff, Inc. (PB). PB was also the Construction Project Management entity. The General Contractor was/is Foulger-Pratt Contracting, LLC (FP). The concrete subcontractor to FP was/is Facchina Construction Company, Inc. (Facchina), with VSTRUCTURAL LLC (VSL) providing post-tensioning shop drawings, hardware, and on-site consultation to Facchina. R & R Reinforcing, Inc. (R & R) installed the mild reinforcing steel and post-tensioning elements for Facchina. Rockville Fuel and Feed Co., Inc. was the ready mix concrete supplier to Facchina.

The construction work was inspected and observed by an independent inspection agency (RBB) under the terms of their Contract with Montgomery County and in their role as Special Inspections Program Special Inspector per the Project Statement of Special Inspections Agreement, as well as Montgomery County Department of Permitting Services (DPS) staff (who made periodic visits) and WMATA representatives. In addition to the structural design, PB also provided “Construction Management services,” which required them to have full-time engineering representation on site. PB also provided site structural inspection, pre-pour, of some of the major deck pours.

During construction, after post-tensioning pop-outs were noted (Attachment 6) and cracking was observed, (Attachment 7) it was noted that various slabs were “thinner” and “thicker” than the Contract Documents specified and that other post-tensioning tendons were exposed. Sometime after those concerns were raised, Simpson, Gumpertz & Heger (SGH) was retained by Shapiro Lifschitz and Schram, circa March 2012 to perform a limited condition assessment. That limited condition assessment apparently included field visits, and limited structural analysis. Our discussion regarding the SGH report(s) can be found hereinafter in this report.

V. DISCUSSION

A Request for Proposal (RFP) for the construction of SSTC was issued for bid and then was supplemented by various pre-construction addenda, etc., yielding the "Conformed Set," which we understand was the Contract Set. During construction the Conformed Set Contract Documents were modified by ASIs (which included, among other things, inclusion of RFI responses, and document changes and/or clarifications). There were also non-Contract Documents RFI responses (asked by FP and their subcontractors), which PB and others responded to. The RFIs, ASIs, sketches, and field directions have, we understand, been incorporated by the Contractor into two red line documents (aka as-builts) prepared by the Contractor dated 2/1/11 (Attachment 8 first sheet only) and 6/15/12 (Attachment 8a, first sheet only).

A. General Observation

Documents which were available to PB during the design process, before construction commenced, and those available to all parties during construction regarding the possibility of cracking of the concrete structure, including actual cracking noted a day or two after a pour on 10/4/10 (Attachment 9), warned of the possibility of cracking and continuing cracks of the SSTC structure.

The Codes and Standards under which the building was to have been designed not only have requirements to limit the stresses induced into the concrete during the introduction of post-tensioning forces (to limit cracking) as described hereinafter, but also contain numerous additional warnings regarding cracking due to "restraint" forces and curing procedures. In addition, it is Industry Standard in post-tensioned structures to limit design "over-balance" (i.e., the upward force due to the initial post-tensioned stressing vs. the dead load of the structural concrete slab) to no more than 80-90% of the actual weight of the beams, girders, and slabs.

The restraint forces and the stresses they induce should have been accounted for in the design of a post-tensioned structure. Restraint occurs in concrete structures when structural elements are prevented by other elements in the building structure from moving, such as when experiencing drying shrinkage and/or temperature changes. Restraint is more prevalent in post-tensioned structures. In the case of the SSTC facility, the poured-in-place post-tensioned structural elements with a relatively thin slab profile are tied to stiff concrete walls with formed shear keys and reinforcing bar dowels without a bond breaker or slip sheet (as would be Industry Standard). This design prevents the slab from moving freely and therefore causes cracking to occur.

Additional restraint-induced cracking was/is created by the very large concrete girders themselves, which are restrained by the large interior concrete columns, exterior columns poured internally with perimeter walls by design, and induced columns with multiple post-tensioned girders framing into them.

Concerns were raised by DPS and the Contractor (May 2010-November 2010) (Attachment 10), and sketches, dated 8/11/2010 (Attachment 10A), were prepared by PB before the first elevated concrete deck was poured to discuss a possible relief of one of the possible causes of future cracking. To the best of our knowledge based on the information provided, the work shown on the sketches was not issued as a Contract Document (they were only marked "SK"). Apparently

the bond breakers were not installed and, in our opinion, would not have dealt with the majority of the restraint issues inherent in the design.

B. Construction Process Outline

We offer the following outline to describe the probable construction process, which we cannot confirm in all instances, by the reports presented, was followed specifically on this Project. (This construction process outline is offered to allow the reader to understand a generally-followed construction process).

One of the early events as part of a construction project is for various contractors to submit shop drawings and make other submittals, which are documents that implement the Project design as shown on the Contract Documents. Those shop drawings provide details of exactly how each and every element is to be fabricated and installed with sufficient detail to allow the facility where the fabrication or mixing of those elements is to occur to fabricate or mix the materials and to provide descriptions to the field personnel with instructions on how and where to install those elements to conform to the Contract Documents.

The submittals would have been sent through "channels" under the procedures prescribed in the Contract Documents for review and approval of the appropriate design professionals and others.

Regarding the concrete mix submittals, the Contract Documents indicate generally minimum and/or maximum quantities of materials to be used in order to achieve the as designed strength and durability, but leave it up to the Contractor to submit, for approval, the detailed mix designs.

A slab construction joint submittal was made and approved on the Project. (Note: the actual pour sequence numbering on that submittal was not necessarily poured in numerical sequence.) This submittal also indicated the one North and the one South expansion joint and East and West side construction Pour Strips (Attachment 11). We would note closure pours (aka Pour Strips) are not expansion joints in the finished slabs, they simply are installed to allow a portion of construction phase shrinkage to be accommodated.

We would note that changes to the Contract Documents are not, per Industry Standard, to be made on submittals (which was done on SSTC by PB). The comments on the submittals are supposed to be only those to ensure the shop drawings indicate the details of elements needed to achieve the design requirements that are shown on the Contract Documents. If PB noted "issues" when reviewing the shop drawing that, in fact, were "errors or omissions" of the Contract Documents prepared by PB, the changes required were not to be made on the shop drawings, but were to be issued via a Contract Document change such as an ASI.

To summarize the probable actual construction process in abbreviated terms:

> The concrete deck construction would be accomplished by installing temporary formwork, followed by bottom mild steel reinforcing bars, followed by profiled post-tensioning conduit (with encapsulated unbonded unstressed wire) with the

necessary anchorage and stressing ends affixed to the cables and the stressing end free in the stressing anchorages.

This would be followed by, and at times installed at the same time as, the placement of items embedded in the concrete for other trades, including, but not limited to, plumbing and electrical, (indicated generically on the Contract Document plumbing and electrical drawings), and finally the placement of the mild steel top reinforcement.

After inspections, the concrete would then be poured, usually with the material placed into a hopper of a concrete pump to be pumped to the point of deposit. (At times on this Project, concrete was transported by a concrete bucket suspended from a crane, and for one Pour Strip, directly from the trucks with PB approval.)

The concrete would then be screeded and finished to get to the Contract Document-specified elevations and finishes.

The concrete would be sampled at the point of deposit and observed to measure its characteristics and allow for testing of laboratory and field cured cylinders.

What are noted as field cured cylinders “FC” would be cured on site in the same manner as the slabs (i.e., not in curing sheds) and lab cured cylinders would be cured in heated or cooled (weather dependent) sheds. After the Contract Document-required on-site curing time, the cylinders would be transmitted to the lab for additional curing and testing.

Depending on weather conditions, the concrete decks would be cured and protected based on the Contract Document requirements (cold weather would apply to concreting operations for major portions of this Project).

After the concrete was poured and achieved a certain strength based on cylinder strength test results, the tendons (cables) were pulled (jacked) to a predetermined as-designed force and locked off (wedged). The “elongation” of the tendons/cables/strands was noted and compared with the as-calculated elongations. After the requisite force was achieved and the tendons were wedged in place, with approval of the Structural Engineer of Record for the building structure, the loose end tendons (cables) would be cut, the formwork removed, reshores (vertical temporary members) installed under that concrete just stressed (if required based on the in situ concrete strength based on cylinder strength and approved formwork shop drawing requirements) and formwork placed for the slabs above (resting on the poured slab below).

The post-tensioning conduits were grouted after stressing. This grout provides continuous bond between the surrounded strands, which provides an additional factor of safety from “catastrophic” collapse beyond that provided by the anchoring of the wires at the ends.

C. Discussion

We would note the Contract Documents indicate when there is a discrepancy between the Contract Document requirements, the most stringent requirement applies.

The submitted post-tensioning shop drawings indicated the number of tendons (a post-tensioning tendon consists of 7 high strength wires braided together to form one tendon) and the amount of force (jacking force) that, by the requirements in the Contract Documents, had to be introduced into the tendon. That force was induced via a hydraulic ram (e.g., jack) attached to the tendon to achieve the “effective force” called for on the Contract Documents.

We would note that given the tendon profiles as called for on the Contract Documents, numerous changes were made in the field as the profiles could not be achieved due to the design geometry and configuration, nor could the effective forces be achieved with the profiles called for on the Contract Documents.

The jacking force is essentially the numerical addition of net effective force as required by the Contract Documents, plus the losses that occur during the stressing operation due to materials used (as allowed per the requirements of the Contract Documents), e.g., frictional losses, creep, wobble, modulus of elasticity (a variable with a minimum strength), etc.

The necessary jacking force calculation was then used to determine how long the cable was to be extended under the force and that extension, as well as the force, was measured in the field (i.e., elongation). There was a tolerance allowed in the measured elongation originally limited to 5% by the Construction Documents, but changed to 7% by PB.

The jacking force was induced after the concrete had achieved sufficient strength based on the site sampled concrete cylinders. (NB: the Contract Document drawings require 75% of f’c (where f’c represents the concrete compressive strength in psi at the time of initial stressing) or 6,000 psi, yet the Contract Document specifications allow stressing to begin at 4000 psi for 50% of the tendons.)

The Contract Document drawings have a stressing sequence noted. There was a change made to that stressing sequence after several deck pours, as approved by PB.

The Contract Documents also require that stressing of 50% of the post-tensioning be accomplished within 96 hours of a pour. We can find no information in the RBB reports to determine if those limitations were followed.

The Contract Document specifications indicate certain allowable materials and the allowable quantities of those materials that are acceptable to be used in the concrete for the Project.

Concrete mix designs were submitted based on those requirements.

The approved concrete mix design used for the 8,000 psi concrete in the deck, slabs, girders, and beams was 8K2DC2NL, the details of which were included in a concrete mix submittal (Attachment 12). Although another 8,000 psi concrete mix (8K2DC4NL) was submitted, we cannot find reference in RBB reports as to where it was placed. The 2NL mix was approved by

PB, WMATA, and others after numerous iterations. We believe notations by the contractor on the submittal that the ground granulated blast furnace slag (GGBFS) (part of the cementitious material) would cause the concrete to achieve considerable strength, "later" was advising the reviewer that it would take longer than 28 days to achieve full strength. PB Review comments on the referenced submittal remind the contractor, notwithstanding the contractor's comments, that the mix had to achieve the strength required before stressing per the Contract Documents.

The approved concrete mix included, as allowed by the Contract Document specifications, slag as part of the cementitious material.

We do note (from the Federal Highway Administration publications website):

> *The compressive strength development of slag concrete depends primarily upon the type, fineness, activity index, and the proportions of slag used in concrete mixtures. In general, the strength development of concrete incorporating slags is slow at 1-5 days compared with that of the control concrete. Between 7 and 28 days, the strength approaches that of the control concrete; beyond this period, the strength of the slag concrete exceeds the strength of control concrete (Admixtures and ground slag 1990). Flexural strength is usually improved by the use of slag cement, which makes it beneficial to concrete paving application where flexural strengths are important. It is believed that the increased flexural strength is the result of the stronger bonds in the cement-slag-aggregate system because of the shape and surface texture of the slag particles.*

and

> *Problems occur when slag concrete is used in cold weather applications. At low temperatures, the strengths are substantially reduced up to 14 days, and the percentage of slag is usually reduced to 25-30% of replacement levels; when saw cutting of joints is required, the use of slag is discontinued (Admixtures and Ground Slag 1990).*

A comparison of some of the characteristics of the approved 8,000 psi concrete (8K2DC2NL) mix vs. the Contract Document requirements follows:

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Table 1.

	Air Content	Water/Cementitious Material Ratio (by Weight)	Slump	Slag	DCI
Contract Document drawings	6% ± 1%	Not listed	Not listed	Not listed	Not listed
Contract Document specifications	5.5% ± 1.5%	Maximum 0.40	*"4" or 8" for concrete with a verified slump of 2"-4" before a high-range water reducing admixture is added.* " NB: we did not find such a test.	*"…50% including no more than 20% fly ash and silica fume not exceeding 10%."*	"*Where indicated*" (no quantity noted). We note no locations were indicated in the Contract Documents
Approved submittal	5% ± 1.5%	0.29	8"	1.96cf/cy	2 gallons/ cubic yard
WMATA	Not listed	Not listed	2"-4"	Not listed	Varies from 4 to 3.5 gallons based on water/ cement ratio

Apparently not all of the RBB inspectors had the certificates required of them by the County, although they were apparently very experienced in construction in general. We have no information regarding the post-tensioning inspectors' experience or PTI certifications required for inspectors.

RBB seemed to rely on approved shop drawings for their inspections though they note they also used the "*design drawings*." We would note that discrepancies between those documents had to be resolved (there were discrepancies between them and, due to those and possibly other reasons, PB made design changes on the shop drawings). We do not know how those discrepancies were resolved.

There was a great deal of correspondence between RBB and PB, apparently via email and telephone, and numerous sketches were generated by PB.

There were numerous deficiency lists generated and referenced by RBB but we have found only three within the items provided.

PB's structural engineer visited the site and reviewed the area of a pour before the pour for a number of deck pours and in each case, PB's structural engineer eventually signed off on them (we can only find eight such reports for the eighteen post-tensioned deck pours, including sub-pours, in the information provided).

There are notations in the RBB reports of cold joints forming in the concrete as shown in jobsite photographs (Attachment 13) (i.e., concrete hardening not at a pre-planned joint) without indications as to the resolution of those unplanned joints, only that stressing was delayed.

In the information provided, we can only find PB approval of formwork removal and stressing record review for limited pours.

We can find only one inspection report in those supplied with notation of formwork inspection required of RBB, and then without detail.

Despite Project requirements, we can find no record of measurement of slab thickness, in situ clear cover determination, or of the deck finishing process in the RBB reports presented to us.

We can find only several notations in the RBB daily reports regarding the concrete curing process used as to methods and/or time, and only limited records of in situ concrete deck temperatures. Also, there are no notations as to above slab wind break installation.

We found no RBB inspection reports of the installation of the evaporation retarder called for or the curing compound called for, nor for any deck curing methods used in the non-winter months.

There are several Project photographs that show workers using procedures that are not approved for winter concreting. Attachment 14 shows a worker spraying something, presumably an evaporation retarder or curing compound, on the concrete slab while walking on it. Another photograph shows a worker using a Rosebud acetylene torch (Attachment 14A) in an apparent attempt to heat epoxy-coated reinforcing before a pour. In a third photograph, a worker is apparently applying some type of deicer on the reinforcing steel.

During the period 10/02/10-10/05/10, cracks began appearing in slabs in areas where no stressing had not yet occurred.

There is an expansion joint called for on the Contract Documents in the center of the ellipse at each side. The distance to the temporary Pour Strip on the East and West end is approximately 240 feet on the centerline of the radius and another 40 feet (280 feet total) on the outside radius. (A Pour Strip is an area of a slab left out during construction and then placed after adjacent concrete has been poured and has had an opportunity to shrink. It is not an expansion joint.) We would note the as-designed Pour Strips themselves are substantially wider than the normal 3-4 foot Industry Standard.

We would note, WMATA, in its Manual of Design Criteria (Attachment 15, defines expansion joints as a minimum of 1" wide and indicates they "*…must have assured free movement…*"). The pour joints therefore are not expansion joints. Therefore, in actuality, the expansion joint

spacing is approximately 580 feet at Level 330 and approximately 315 feet at Level 350 vs. the WMATA Manual of Design Criteria limitation of 100 feet.

During the pouring of the concrete, RBB initially took four composite sets of cylinders (a composite set being defined in the Contract Documents as two cylinders): two sets for tests at three days (one set field-cured and one set lab-cured), two sets for tests at seven days (one set field-cured and one set lab-cured) two sets for tests at 28 days (one field-cured and one lab-cured), and two sets held for possible testing at 56 days (one field-cured and one lab-cured), ("at" meaning to be tested at that number of days from sampling during a pour).

Additional cylinders were taken, we are told, for stripping/stressing decisions.

There apparently were also "companion" cylinders made. We do not know who tested the "companion" cylinders (we have no results). It is generally the concrete supplier who takes and tests them.

The sets of cylinders were to have been taken at the points of deposit (final discharge) for every 50 cubic yards of concrete per class (strength) (i.e., one set for every fifth ten-yard truck) plus one set if a fraction of concrete more than five cubic yards and up to 25 cubic yards was poured.

We understand of the trucks to be tested (1 in 5), the first one that entered the site went directly to the RBB "testing" station at grade, while the next four went directly to the pump.

Thereafter, one in five went to the testing station, and four went directly to the pump. Based on visual observations, we understand, RBB (possibly with the contractor present) determined if water needed to be added. (NB: It is not Industry Standard for the testing agency to determine if water is to be added or in what quantity. It is generally the concrete superintendent.) Then concrete from the truck being sampled was sampled for air content, slump measured, and cylinders taken (unclear if before or after water was added, if it was added at all). RBB also noted the ambient air temperature when each of 5 trucks was sampled. The tested truck then moved to the pump hopper and the concrete was then chuted into the hopper and pumped to the point of deposit.

We understand that RBB was present at the location where other four trucks of the five truck set where the concrete was discharged into the pump and RBB visually determined (see notation above for Industry Standard) if and directed how much water could be added. We do not know if RBB took slumps to make that determination or actually noted the water withheld at the plant vs. that added or on what basis, if they in fact did so, RBB made the determination as to the amount of water that could be added.

The cylinders taken at the testing station were placed in the site "curing shed," apparently heated or cooled as weather required, and left for the appropriate curing time and picked up and transported by RBB to the RBB lab.

We understand "all parties" at some point in time agreed that on a random basis, 3 times per pour (once near the beginning, once in the middle, and once near the end of each pour), i.e., once every 20-30 10-yard trucks, 3 additional cylinders (not known if a set or not) were to be taken at the point of discharge on the deck and left on an adjacent pour to be cured as the

concrete being poured would be cured. There are notations in RBB reports that those cylinders were at times placed in a curing box.

RBB apparently did not test unit weight of the concrete.

RBB was to visit the concrete batch plant but we could find no reports of these visits in the information provided.

The water/cement ratio of the concrete is not noted by RBB on their daily field reports but at times, the reports indicate it was measured, which it cannot be, as it is a calculated value based on material quantities noted on the concrete batch tickets and we have been provided no such calculations.

Cylinders that were field cured (not known if cured on deck or at the testing station) are marked FC in RBB compression strength test reports, but we see no notation to differentiate deck cured vs. testing station cured.

The number of revolutions of the truck being sampled as reported by RBB ranged from ~68 to ~276 (300 is max per ACI and WMATA) before offloading the concrete.

The time from the beginning of mixing to the beginning of the offloading was not noted in the RBB reports (but 90 minutes is generally the maximum time allowed from batch mixing to concrete placement in non-hot weather conditions).

Truck delivery tickets were attached to only a few of the handwritten RBB reports we received, with an even more limited number of batch tickets attached to those reports, making evaluation of certain aspects of concrete material quantities and placement conditions difficult.

During the plant filling of a concrete truck, for ease of transportation, it is possible half of the material was placed and the drum turned a few revolutions, and then the second half of the batch installed. Also for ease of transportation, some of the water specified for the mix can be withheld, with the amount noted on a batch ticket to indicate the quantity of water that could be added in the field while keeping the concrete within the mix design parameters. Since only a few of the batch tickets (which showed the amount of water that could be added in the field) were attached to the field reports that we received, we cannot determine the quantity of water added as compared to what could be added within the approved mix parameter requirements. (We do note there was a notation in the RBB reports of the water gauge on truck(s) being broken at times.)

The RBB reports did note in their daily reports when water was added to some of the one in five trucks they sampled, (we do not know if this was repeated on the next four trucks of the five as we have no reports of observation of those in what was provided). There is no way to determine if the amount of water added, if any, was acceptable without knowing the amount withheld, as there is no indication of the quantity of water withheld without batch tickets or added for many of the one in five trucks and for none of the 80% of the trucks beyond the ones actually tested.

In a limited number of their daily inspection reports, RBB indicated that water was added to the concrete in the truck, but there was, however, insufficient information (i.e., batch tickets) provided to determine the amount of water withheld at the plant.

If more water is added than the quantity "withheld," the concrete as placed can be made substantially "weaker" than the mix design.

RBB daily reports note instances of concrete coming down the chute with clumps of cement and also that trucks were being sent to the "washout" (i.e., material thrown away) because the trucks had been mixing too long or due to the clumps of cement.

Water, when added in the field, is generally added to assist in pumping the concrete and in the process of finishing the slab (i.e., after the concrete is placed concrete finishers move the concrete around to get to the correct elevation and thickness, usually done by sticking the concrete with marked probes (aka sticks) and/or having the elevation "shot" by a surveyor, and then use machines and hand labor to achieve the specified finish on the slab).

RBB also notes on their daily reports at times, wide variability in the amount of entrained air and at times notes "Fritz" packs were thrown into the truck being tested (one in five) to add air.

Because testing was performed on only one of five trucks, there is no way to know what was added to or the cylinder strength for the other 80% of the concrete placed in a given pour regarding air or added water, and in fact, concrete strength. Approximately 7-115 trucks were needed to deliver the concrete for deck pours, depending on the yardage being poured (i.e., typically 10 yards per truck).

However, it is interesting to note in the RBB concrete compressive cylinder break reports indicating the cylinder compressive strengths at the various testing dates, curing days 3, 7, 28, and limited cylinders held for 56 (aka 58), at times cylinders for testing at five days), the water/cement ratio is noted as being between 0.24-0.26, (0.28 is the theoretical minimum water/cement ratio that would be required for 100% cementitious material hydration) but there are several with 09.26 (which we believe is typographical error).

We believe the 0.24-0.26 number may have been an entered default number used in the compressive strength test results form preparation that should have been calculated for each cylinder set based on material quantities taken from batch tickets and after "added" water. The same range of number appears apparently regardless of whether or not water was added in the field when noted in RBB reports. (NB: the approved mix design had a water/cement ratio of 0.29.)

We also believe the .24-.26 range is not consistent with the slumps as RBB reported, presumably after a high range water reducer was added per the approved mix design. In fact if the water/cement ratio was .24-.26, it is our opinion the concrete would have been very difficult to pump and even harder to finish and would not have permitted complete hydration to occur.

For 8,000 psi and higher concrete, the slabs can as long as four to eight hours, especially in cold weather, to finish. During that process "bleed" water is brought to the surface and could affect the entrained air in the top surface of the concrete (approximately thirty millimeters).

We understand that polyurethane and insulating blankets were installed on top of the deck for curing purposes but were not, in some cases, installed until the "end of the day." As a result, several hours would have passed between the time when the concrete achieved its final set and when protection would have been provided. This means that hydration would have been slowed during this time and the concrete could have experienced drying without hydration occurring. This could explain the presence of unhydrated clinker and slag particles in certain concrete samples excised from the deck.

RBB issued "Compressive Strength Test Reports" as the concrete cylinders were broken in the lab, for both the field cured cylinders and the lab cured cylinders, apparently after the prescribed curing time. We do not know if a curing bath was used in the RBB lab, but we are advised that cylinders were cured in a moist room per ASTM 31 and 511.

We would note the RBB Compressive Strength Test Reports indicate essentially all of the cylinder break test results with strengths substantially higher than the minimum compressive strengths required by the Contract Documents.

The concrete cores we excised from the SSTC slabs exhibited lower compressive strengths than the compressive strengths measured from the cylinders that were cast at the time of construction.

We would note ACI 318-02 defines the method to be used for determining acceptability of concrete. Under those acceptance criteria, Pours 1A, 1B, 1E, 1H, and 2A contain unacceptable concrete. Further, Pour Strips are so severely cracked, they also contain unacceptable concrete.

We then used the method prescribed by ACI 214.4R-10 for determining concrete strength, a statistical evaluation allowed by ACI 318 for evaluation purposes with a 10% fractile and with 90% degree of certainty, yielding an overall deck in situ concrete strength of 6,970 psi.

There are a number of different factors that can cause a difference in strength between those measured in cylinders taken from concrete as it is placed in a structure and those taken from concrete core samples taken at a later date (beyond additional cure time):

Variability of water/cement ratio

It is well established that compressive strength of concrete and the water/cement ratio of concrete are interdependent. When water is added to the concrete (increasing the water/cement ratio), the compressive strength is reduced. The RBB daily reports indicate the concrete used for cylinders was taken for the most part from the concrete trucks prior to pumping and was not sampled from the deck at the pump discharge. In addition, testing reports are not clear as to whether water was added to the mix after the concrete sample was taken. The actual amounts of water added to the mix at the time of construction and after the concrete was sampled are not known, except of limited trucks. As a result, concrete placed on the SSTC deck would have a differing water/cement ratio than that of concrete being used to make the concrete cylinders, and consequently, compressive strengths that would be

lower than that of the cylinders. Furthermore, only one in five trucks was sampled during construction, which is consistent with Industry Standards. Nevertheless, cylinders were not made for a majority of the concrete placed on the decks. As a result, concrete that was excised from the slabs via coring may not necessarily correspond with cylinders that were made for a specific concrete placement.

Cold Weather Construction

Concrete must be placed in an environment that is warm enough for the chemical reaction that gives concrete strength (known as hydration) to occur. The Project inspection reports indicate that portions of the concrete slabs were poured in temperatures below 32°F. Concrete placed in these temperatures is at risk to either freezing or slow development of strength. Construction records indicate that the contractor heated the slab from the level below, provided thermal breaks around the slab formwork below, and then placed plastic sheeting over the top of the slab with insulating blankets over the plastic sheeting. There is no record of wind breaks above the slabs.

A number of pours were started and continued when temperatures were less than 32°F and as low as 15°F, based on RBB reports.

Inspection records received indicate that concrete temperature on the deck was measured for several pours and only at day 3 by means of a thermometer placed on the top surface of the concrete slab (under the plastic sheeting). The internal temperature of the concrete was not monitored during placement based on the reports provided, or subsequent days before the temperature measurement on day 3. It is unknown what the internal temperature of the concrete in the deck was during the hours after the concrete was placed to the time when finishing was completed and insulation began. We do know the ambient temperature at the beginning of pours and at every fifth truck.

Petrographic examinations of the concrete cores from the slabs indicate that between 5-12% of the Portland cement and 16-18% of the slag was unhydrated. This is consistent with concrete experiencing a temperature low enough to slow hydration to the point that the available water dried out before the cement and other cementitious materials could hydrate. The curing method provided by the Contractor was, when temperature required for cold weather concrete, apparently a dry heat method, and no special precautions are noted in the construction records beyond providing the plastic, sheeting and blankets on top to prevent drying out the concrete. ACI 306 Section 8.2 notes that during "very cold" weather, it is necessary to add the moisture to the heated air to maintain relative humidity and to have plastic curtains below the slab to retain the heat.

ACI 306 also states that a structure with SSTC's service category should be protected for 6 days when Type I or II cement is used, unless an additional 100 lbs/yd^3 of cement is used in the mix, of which we have no record. In that case, protection can be removed after four days. Construction records indicate that

plastic sheeting and insulation were removed from the structure after 3 days on one pour and an unknown time for other pours as reported in the information provided. By removing protection early, hydration of the concrete would have been slowed or stopped, which would explain the presence of unhydrated cement and slag.

In contrast, the concrete cylinders made from the concrete as being placed after being molded were stored in a curing box at the testing station, and therefore experienced a substantially different environment than the concrete in the slabs. ASTM C31 states that during initial curing after molding, specimens to be lab cured are to be stored in a temperature range between 60 - 80°F and in a moist environment for up to 48 hours. The temperature in these boxes would likely have been higher than those in the deck and the moisture conditions unknown, and concrete hydration process of the cylinders would have continued to progress while concrete on the deck slowed or stopped after protection was removed. Furthermore, after the cylinder molds are stripped, ASTM calls for the cylinders to cure in either water baths or in a moist room. This standard practice would again continue hydration of the cylinders during the period after protection and curing was removed from the deck, while the deck concrete was not curing under similar conditions.

Curing

Wet curing of concrete, such as by wetted burlap mats, was not applied to the SSTC deck. Impermeable sheeting was placed over the concrete to limit moisture loss from concrete due to evaporation. Evaporation retarder was sprayed on the deck, but we do not know if that occurred before or after impermeable sheeting and protection were removed.

Standard curing is required by ASTM C31 for acceptance testing for strength. Thus, the cylinders should have been stored either in water baths or a moist room after being removed from curing boxes and sheds and shipped to the lab, where additional hydration would have occurred. Moist curing, which promotes concrete hydration that the cylinders would have experienced could have resulted in compressive strengths that would have been higher than that of the concrete in the decks.

Compressive strength testing of concrete specimens removed from the structure has yielded a wide range of results. The composition of any concrete varies (nonhomogeneous), and a concrete's compressive strength can be affected by numerous factors that start, as noted earlier, with variability in manufacture and procurement of cementitious materials and aggregate, mixing and transportation, addition of water on site in varying amounts from truck to truck, placement and finishing, ambient conditions at the time of placement, and curing.

At SSTC, the supply side variables (raw materials and plant mixing) have been considered to be fairly consistent since we have no information that tells us

differently. Petrographic examination of the hardened concrete of samples excised from several different locations indicates that the concrete aggregates and GGBFS are generally consistent. This suggests that the variation in compressive strengths from pour to pour occurred due to differences in placement, curing, or a combination of on-site factors. It is not known what amounts of water were added to each truck that came to the site. The addition of water could significantly affect compressive strengths within a given placement. This variation is further complicated by the fact that varying amounts of water would have been added from truck to truck on pours that occurred on different days with different ambient conditions and potentially different finishing times. Due to the very cold weather during many of the pours, temperature variations and variations in the time it took to apply cold-weather protection could have had a substantial impact on final concrete strength.

In addition, petrographic examinations consistently have identified instances of entrapped air (as differentiated from entrained (added) air) within concrete samples. Entrapped air is air bubbles that are randomly introduced into the concrete during mixing and placement and then are not consolidated by vibration during construction. These air voids create weak spots in concrete that are random, of varying size, and with no discernible pattern as to their location or frequency of occurrence.

The compressive core strength data follows no clear statistical pattern with a wide standard deviation. Furthermore, the data does not indicate correlation between strength and the various pours. There is no individual pour that was statistically more uniform than the other pours, or consistently stronger or weaker than one pour or the other.

The apparent randomness of the strength values achieved in the completed structure are suggestive of insufficient and inconsistent quality control in cold weather working conditions that were adverse both to the concrete and the workers. Specific contributing factors may have included:

- Areas where concrete was left exposed longer than others before curing blankets were placed
- Excessive movement of concrete during placement and finishing
- Delayed finishing
- Inconsistent heating during curing

  and/or
- Arrested hydration.

Minor variations in the testing procedures used by our four selected materials testing laboratories may account for minor differences between data sets provided by those different laboratories, but the in situ concrete strengths at SSTC are below the Contract Document requirements. Based on our review, all laboratories complied with ASTM C-42 testing protocol, and we are not aware of specific variations that may have affected test results.

As noted earlier, we are told taking three sets of cylinders on the deck at random times during a deck pour was agreed to by the parties. That was initiated, though we cannot verify when, based on the RBB reports, yet Montgomery County DPS noted that, for the concrete sampled in Pour IC, only two sets were being taken on the deck. The concrete cylinder strengths, with only two sets of cylinders taken on the deck, were approved by PB.

The RBB daily handwritten reports were followed by typed daily reports and then general monthly summaries.

The handwritten daily reports have numerous notations of deficiency lists of work to be corrected prior to concrete being poured, but we have seen only three in the information provided.

The various RBB reports also indicate a structural engineer from PB visited the site before pours and issued reports and approval, but we have only seen eight of those reports in the information provided.

"Pour cards" (i.e., "sign off" approvals were prepared and initialed by RBB and in some cases subcontractors) to indicate a deck was ready for a pour (we have only seen several of those reports in the information provided).

"Tendon count forms" were produced by RBB to verify the correct number of post-tensioning tendons had been installed correctly. However, we have only seen several of those reports in the information provided.

We understand the stressing elongation reports were submitted to PB for approval prior to cutting the post-tensioning tails and subsequent grouting, but we have only found limited reports of such approval in the information provided.

The Contract Documents require shoring under beams in the same line to remain until 100% are stressed but we did not find documentation to indicate that that was done, though we understand stripping of forms was "approved" by PB.

While the Contract Document drawings note a maximum permitted pour size of 12,000 square feet and pour length of 130 feet, the approved construction joint layout shop drawing yielded pours where pour sizes slightly exceed these limits in both square footage and length.

We would note some sub pours of slabs (i.e., IEa, IEb, 2Ia, 2Ib) were required for reasons unknown. We find no submittal approval of those in the documents provided.

The Contract Document drawings also require formwork to remain in place until the concrete achieved 75% f'c (6000 psi), but this seems not to have always been done based on RBB reports (though with PB approval, apparently based on cylinder strength test results).

Note: the use of cylinder test results to determine early concrete strength for stressing and formwork removal (where taken) may have indicated a strength where formwork stripping and stressing could occur but the cylinders may have recorded incorrect results for the reasons noted herein.

The Contract Document drawings indicate clearance (top cover) over slab and beam stirrups and post-tensioning to be 2" with no mention of tolerance, however PB later indicates the tolerance per ACI was acceptable and later advised a lesser number was acceptable for tendon clearance, apparently due to lack of space for placement all the as-designed elements.

The Contract Documents indicate that the slab and beam depths shown are minimum and that additional thickness/depth to accommodate drainage slopes are to be poured monolithically.

This would override the ACI slab tolerance regarding "over" thickness above that shown on the Contract Documents (i.e. greater than 10) as confirmed by the design team response to RFI 216 and 571, (Attachment 16), which questioned the slope of the slab at certain locations. The response indicated the need for "cambering," e.g., crowning, the slab, graphically indicating only the top of the slab "cambered," not the bottom, which would therefore yield slabs of greater than 10" in thickness in certain locations.

VI. AS-DESIGNED

A. Analysis Methods

Our analysis methods included the use of ADAPT-PT© v.8.0, ADAPT-EDGE© 2012, and CSI Bridge© v.15.2 and hand calculations. Our work was performed using the requirements of IBC 2003, ACI 318-02, PTI Specifications, and the WMATA Manual of Design Criteria and Standards.

As those who use computer programs understand, there are always options in the way a user (in this case, engineer) can allow a program to run, such as using default values, and more importantly, determining parameters that are at the choice of the engineer. These include, but are not limited to how various portions of a structure are modeled insofar as their attributes as they affect other elements.

We have made the engineering decisions in this regard as to the torsional analysis, column evaluation, etc. (those decisions are made based on Industry Standards, such as ASCE, and engineering experience).

According to ACI 318-02 (Attachment 17, cover sheet only), three loading conditions (limit states) must be analyzed in the design of the Project:

- Initial - includes unfactored concrete self-weight and post-tensioning forces.
- Service – includes all unfactored loads (concrete, self-weight, superimposed dead load, live load, snow load, and lateral load) and post-tensioning forces.
- Ultimate - includes all factored design loads (concrete self-weight, superimposed dead load, live load, snow load, and lateral load) and hyperstatic forces.

1. CSI Bridge© Models

Includes moving load analytical capabilities, and was used as a "bridge," as an analytical tool. These models were used to determine the beam, girder, and column internal force envelopes (i.e., maximum shear, torsion, and moment based on the application of the required loads). The moving truck loads were applied to the structure using one or more predefined paths that simulate the specified slab marking/lane plan shown on the Contract Documents. (NB: For clarity throughout this report, the words "truck" and "bus" may be used interchangeably as AASHTO references trucks only, but the design requirements are also used for buses.) The resulting force envelopes were used to verify that our ADAPT-PT© analyses and hand calculations utilized the worst-case loading conditions.

2. ADAPT© Models (post-tensioning design and analysis)

a. <u>ADAPT-PT©</u>

This program allows the user to define element material properties, geometry, and loading conditions. Our results included, but are not limited to, concrete stresses under initial and service loads, ultimate strength limit requirements, post-tensioning force requirements, and mild steel reinforcing bar requirements.

b. <u>ADAPT-EDGE© Model (Finite Element)</u>

ADAPT-EDGE© 2012 is a three-dimensional, finite element, post-tensioning analysis and design program.

We used ADAPT-EDGE© to calculate internal concrete stresses (prior to cracking) in the post-tensioned slabs for areas of the structure with complex geometry. Note that this software assumes two-way bending and therefore was not used to predict one-way slab behavior at ultimate (factored) load levels.

3. CSI Column

CSICOL© V8.3.2 is a column analysis and design program that allows for customization of the vertical reinforcing bar pattern.

B. <u>As-Designed Analysis</u>

1. Applicable Codes and Manual of Standards

The Contract Document drawings reference, among others, the Building Code (IBC 2003), ACI 318, the WMATA Manual of Design Criteria, the WMATA Standards, and the American Association of State and Highway Transportation Officials (AASHTO) *Standard Specification for Highway Bridges, 17th Edition, 2002*. While the Contract Document drawings reference the AASHTO truck load, they do not explicitly reference the "AASHTO Standard Specification."

Relevant Code and design load requirements follow. (Note that the Contract Documents indicate that the most stringent criteria shall apply.)

IBC 2003 (Attachment 18, cover sheet only due to copyright restrictions) notes the Code's purpose is to provide minimum requirements – e.g., "…*this comprehensive Building Code establishes minimum regulations*…" - and "…*prescriptive and performance-related provisions…"* to be used by the structural engineer designing a building.

a. Controlling load combinations listed in American Concrete Institute 318:

   Building Code Requirements for Structural Concrete (ACI 318-02) includes but id not limited to 1.0xD, 1.4xD, 1.2xD+1.6xL+0.5x(L or S), and 1.2D+1.0E+1.0L+.075. Note that D is dead load, L is live load, S is snow load, and E is earthquake or seismic load. F

b. ACI 318-02 contains numerous relevant design requirements including but not limited to the following.

   i. Initial extreme fiber concrete stress in tension is to be limited by design to 3√f'c during the initial stressing.

   ii. Service level extreme fiber stress in tension is to be limited to 7.5√f'c. ACI 318-02 defines behavior classifications and provides extreme fiber stress limits for each classification. The classifications are based on whether the concrete will be uncracked (Class U), somewhere between uncracked and cracked (Class T), or cracked (Class C) while in service. Stress limits for Class U, Class T, and Class C are 7.5√f'c, 7.5√f'c to 12√f'c, and greater than 12√f'c, respectively.

      Based on our review of the PB calculations and ACI 318-02 requirements, we believe that the structure was considered by design to be "uncracked" (Class U) with an extreme fiber tension stress limit of 6√f'c in service load conditions.

   iii. ACI 318-02 notes, among other requirements:

      a) Certain load factors
      b) Analytical procedures to be followed
      c) Allowable stresses

2. WMATA Manual of Design Criteria and WMATA Standards

The Manual of Design Criteria Volumes 1, 2, and 3 (aka Revision 5/6) are dated November 2003 with various individual sheets noted with other dates.

a. Manual of Design Criteria

i. Indicates that for structural members not carrying transit loads, stress and as enforced by WMATA shall not exceed 6√f'c. NB: the Industry Standard definition of post-tensioned concrete is that it is a form of prestressed concrete (see ACI definition hereinafter).

ii. Illustrates HS25 design truck for "Roadway Loading on Highway Bridge Structures" in Figure V.18. Notes on the PB Contract Documents indicate that the HS25-44 truck was the design truck. The truck referenced by WMATA, as noted in Figure V.18, is the same as the AASHTO HS25-44 truck. Figure 10 in the Manual illustrates the truck referenced in the WMATA Design Manual (Attachment 19). Attachment 20 indicates truck loading layouts used in the analysis.

iii. "*Extreme fiber stress in tension* […] *shall not exceed 6√f'c*…" (Attachment 21).

iv. Indicates amount of corrosion inhibitor (Attachment 22).

b. WMATA Standards (in part)

WMATA Cast-in-Place Structural Concrete specification section 03300 dated 03/03 notes (among other limitations noted elsewhere in this report) items that are not called for in the Contract Documents, including, but not limited to:

i. Unit weights to be measured in field
ii. 7-day heat to at least 55 degrees F
iii. Not pouring concrete in less than 40 degrees F and rising
iv. Specific curing methods
v. Core drilling if required for strength evaluation
   a) Taken per ASTM C-42 and concrete must be at least 85% of f'c

3. AASHTO Specification

a. The PB documents reference an AASHTO HS25-44 truck and 33 percent impact factor. The AASHTO Standards define the truck geometry and loads. While not an AASHTO standard, stresses should have been determined by influence line analysis to determine "worst case" loading.
b. Our analysis load cases include not only the load arrangements noted in the PB calculations but also other load arrangements based on those we considered to be critical for proper analysis.
c. IBC 2003 does not reference AASHTO specifically but does reference AASHTO trucks, while the Contract Documents do reference AASHTO truck loads. Based on our review of PB's calculations, the AASHTO truck wheel loads were applied as a concentrated load and arranged in patterns, but we did not find an influence line analysis to determine load distribution.

NB: AASHTO has different criteria and limits for post-tensioned concrete design when compared to ACI 318-02; therefore, we reviewed as to the most stringent.

4. Contract Document drawings dated January 7, 2008, e.g., "Conformed Set."

The Conformed Set of drawings apparently includes all "changes" issued during the bid and negotiation process, as noted earlier, and was used ostensibly as the "For Construction" set.

a. Dead Loads

The Contract Document drawings note a provision for an area of future superimposed dead load of 35 psf in the drive aisles. This apparently is an allowance for a future overlay and other measureable items (if this allowance was used for overlay it would be approximately 3 inches thick). Note that according to the WMATA Manual of Design Criteria, the drive area should be designed for an additional load of 25 psf for a future 2-inch wearing surface.

The addition of either a 2-inch or 3-inch overlay would reduce curb height and would require modifications to the access points to the pedestrian area walkways, elevators, stairs, escalators, etc., including, but not limited to, full perimeter railings, cut in ramps, etc., to meet IBC 2003 accessibility requirements.

The pedestrian area walkway slab dead load was assumed by PB to be 90 psf. In our analysis, we used a superimposed dead load of 75 psf in accordance with the as-designed height of the curbs and the unit weight of the stone concrete as shown on the Contract Document drawings.

b. Snow Loads

The ground snow load is listed on the Contract Documents is 30 psf. Following methodology contained in IBC 2003 to develop roof snow loads, the Code required snow load on the elevated slabs is approximately 25 psf plus allowance for drifting. Note that drift weights vary depending on the height of the parapets, railings, etc.

Also note that there does not appear to be a specific load included in the design to accommodate piling up of snow on the elevated decks.

c. Pedestrian Live Loads

The Contract Documents define the pedestrian area walkway live load as 150 psf. Note that this conflicts with IBC Table 1607.1, Item 32, which requires 250 psf. However, the 150 psf load complies with the WMATA Manual of Design Criteria, which references 150 psf. For our as-built analyses, we subjected the pedestrian areas to a uniform live load equal to 250 psf. Note that this assumption addresses the possibility of a "design" truck driving on the curbs.

C. As-Designed Parameter Discussion

The Project Design Requirements as noted on the Contract Documents, include, but are not limited to IBC 2003, ACI 318, AASHTO, PTI Specifications, the WMATA Manual of Design Criteria, and WMATA Standards.

Review of the Contract Documents and the PB calculations presented show that PB attempted to comply with WMATA's $6\sqrt{f'c}$ extreme fiber tension stress limit for service loads. However, no initial stress review appears to have been performed. Note that ACI 318-02 requires initial extreme fiber stresses in tension to be less than $3\sqrt{f'c}$ by design. Exceeding the initial extreme fiber stress limit, could lead to concrete cracking during initial stressing. The initial cracking would affect the distribution of service level stresses but does not impact the ultimate strength of the structure.

The SSTC facility was designed by PB using a design method of ultimate strength design known as Load Resistance Factor Design (LRFD). This method ensures the reliability of structures to carry load by increasing the anticipated design loads by factors referred to as load factors, and reducing the available ultimate strength of the materials by factors known as resistance factors (aka phi Φ factors). When using this method of design, the designer must use the combination of load factors with the associated resistance factors.

Load factors are Code required multipliers of the actual weight of the structural components (dead loads), added permanent "dead" loads (sidewalks, ceilings, etc.), live loads (people, vehicles, etc.), and lateral loads (wind and seismic) to be used in the design formulas with ultimate material strength when designing a concrete structure.

Phi factors are reduction factors that limit the allowable material strengths used to resist the loads (factored loads) to determine the forces and stresses in concrete.

The base load factors that were used by PB (1.4*D+1.7*L) are based on the requirements of ACI 318-02 Appendix C. This results in factored design loads that are higher than those required by ACI 318-02 Chapter 9 (1.2D+1.6L+0.5S). However, the effect of the higher design loads considered by PB is offset by the increased reduction factors (phi factors) they used that correspond to the Appendix C load factors. Design by the requirements of either Chapter 9 or Appendix C is accepted by ACI 318-02. The load combinations that were used by PB appear to be appropriate.

PB summarizes wind and seismic design load criteria on the Contract Documents. In general, the information on the drawings is consistent with the IBC 2003 and ASCE 7 requirements.

PB uses importance factors of 1.0 for wind and 1.15 for seismic. The design seismic loading in the cases that are evaluated controls over the design wind loads and therefore, since the worst case load combinations are to be used for design, the seismic loads with dead and live load formulas control, not the formulas with the wind component.

D. Additional Factors Contributing to Cracking

1. Initial and Service Level Stress

The design by PB does not limit initial stresses to less than 3√f'c. During construction as a part of the review of R&R stressing records, PB performed several calculations and in 2012 indicated that the stresses (based on the measured elongations) in the structure exceeded the 6√f'c stress limit under service loads. This analysis by PB is another clear indicator that, at a minimum, the structure at particular locations would also be susceptible to cracking under service loads.

2. Lack of Slab Release Connection Details at Stiff Element Interface

"Restraint-to-shortening" (RTS) cracks are caused when slabs are tied to stiff structural elements. The large girders and columns on this Project provide substantial restraint, as do the integral concrete walls.

These elements prevent and/or restrain the normal movement of slabs in conventionally reinforced structures and much more importantly, in those reinforced with post-tensioning, after pouring, before stressing, after stressing, and under thermal changes, etc. As the slabs shrinks due to normal elastic shortening, shrinkage, creep, and temperature, RTS will attempt to stop the slabs/beams from moving freely and they crack.

From "PTI Guide for Design of Post-Tensioned Buildings:"

> "*As the slab is held at the different locations of horizontal restraint while trying to shorten to the portion of the slab near the stiff element will crack away from the unrestrained slab to attempt to release it from this stiff element* [...] *Post-tensioned slabs require more concern regarding RTS cracking because post-tensioned slabs by design have substantially less mild reinforcement than a non-post-tensioned slab and there is not enough mild reinforcement alone to control early shrinkage cracks. If a crack(s) develops due to restraint, a post-tensioned system doesn't have the continuous reinforcement to resist and/or minimize the crack.*"

This structure has long concrete walls below Level 330 and in parts of Level 350 that vary in thickness (8", 24", 32", or 36"). Only two bays of the one level tall, 8" thick infill wall are "separated" from the beams girders and slabs. There is no release connection (slip joint) detail on the Contract Documents for any wall/slab, wall/beam, or wall/girder interfaces other than at the one short length (± eighty feet) of the 8-inch wall.

The large concrete columns and girders also induce substantial restraint and do not allow shrinkage and/or thermal movement of the slabs, beams or girders poured integral with them (as designed).

In Section 7.12.1.2, ACI 318-02 Commentary states: the

> "…*it may be necessary to increase the amount of reinforcement normal to the flexural reinforcement* […] *Top and bottom reinforcement are both effective in controlling cracks. Control strips* [i.e., Pour Strips] *during the construction period, which permit initial shrinkage to occur without causing an increase in stresses, are also effective in reducing cracks caused by restraint."*

The slab elements themselves are also forced to crack additionally to accommodate the normal elastic shortening under creep, shrinkage and temperature change, as well as post-tensioning induced stresses.

3. Typical One Way Slabs between Beams

The Contract Documents indicate required shrinkage and temperature mild steel reinforcement in slabs parallel to and between beams, temperature reinforcement (parallel to the beams) and that post-tensioning cables are allowed to be supplemented by mild reinforcing over the girders between the "tee beam" flanges (the slab itself) from each face of the beam side. At the East end of both level 330 and 350 full length mild reinforcement is called for.

The Contract Documents also specify distributing 1/4 of beam top reinforcing into the tee beam effective flange width each side of the beams. (The tee section of a "tee beam" is used in design, i.e., a part of the slab is used to resist the various forces induced into the beam.)

The approved mild steel shop drawings that RBB used to inspect from do not show this reinforcing, while the Contract Documents do.

Note, per ACI, post-tensioning is "…*a method of prestress in which prestressing steel is tensioned after the concrete has hardened*…" Industry Standard is that prestressing vs. post-tensioning terminology is used almost interchangeably.

ACI318-02 section 7.12.1.1 allows, in design, using either mild steel reinforcement or prestressing steel as shrinkage and temperature reinforcement.

ACI318-02 section 7.12.1.2 states "*Where shrinkage and temperature movement are significantly restrained, the requirement of 8.2.4 and 9.2.3 shall be considered."*

ACI 318-02 Section 8.2.4 states *"Consideration shall be given to effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and unequal settlement of supports."*

ACI 318-02 Section 9.2.3 states *"Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service."*

4. WMATA Manual of Design Criteria and WMATA Standards:

1. *"Crack control and waterproofing of Above-Ground Structure for station mezzanine, platform, track support, ancillary structures and parking structures."*

and

Maximum spacing of expansion joints 100' locate joints judiciously to ensure that electric and mechanical rooms are not impacted."

and

Expansion joints are to be minimum one inch and allow free movement.

Extreme fiber stress is limited to $6\sqrt{f'c}$.

The WMATA Standards also call for 4 to 3.5 gallons of corrosion inhibitor (DCI) per cubic yard of concrete based on the water/cement ratio.

5. Dead Load Balancing

There is no Code requirement as to the limits of dead load balancing percentage, but the *Guide for Design of Post-Tensioned Buildings* issued by the Post-Tensioning Institute indicates "…*balancing more than 100% of the dead load should generally be avoided, as this may result in undesirable cambers, additional cracking, and increased volume changes"* (*ACI Committee 362 1997*).

It is Industry Standard to limit dead load balancing (i.e., upward force due to post-tensioning) in post-tensioned design to 80-85% of slab weight to prevent upward movement during initial post-tensioning and induced cracking.

Our analysis of the as-designed system indicates the balancing of dead loads substantially exceeded these limits.

We would note the Contract Documents indicate, "…*for formwork design, only assume 75% of the structure dead load is balanced by the tendons…"*

6. Concrete Protection and Curing

The Contract Document Specifications specify the "…*work must be installed to comply with ACI 306.1 for cold-weather protection and ACI 301 for hot-weather protection during curing to protect freshly placed concrete from premature drying and excessive cold or hot temperatures*…"

NB: italicized numbers and letters below are as marked in the particular specifications (Contract Documents) quoted below; hence they do not align with the numbering system used in this report.

A. "General: Protect freshly placed concrete from premature drying and excessive cold or hot temperatures. Comply with ACI 306.1 for cold-weather protection and ACI 301 for hot-weather protection during curing.

B. Evaporation Retarder: Apply evaporation retarder to unformed concrete surfaces if hot, dry, or windy conditions cause moisture loss approaching 0.2 lb/sq. ft. x h before and during finishing operations. Apply according to manufacturer's written instructions after placing, screeding, and bull floating or darbying concrete, but before float finishing.

C. Formed Surfaces: Cure formed concrete surfaces, including underside of beams, supported slabs, and other similar surfaces. If forms remain during curing period, moist cure after loosening forms. If removing forms before end of curing period, continue curing for the remainder of the curing period.

D. Unformed Surfaces: Begin curing immediately after finishing concrete. Cure unformed surfaces, including floors and slabs, concrete floor toppings, and other surfaces.

E. Cure concrete according to ACI 308.1 [sic]*, by one or a combination of the following methods:*

1. Moisture Curing: Keep surfaces continuously moist for not less than seven days with the following materials:

a. Water.
b. Continuous water-fog spray.
c. Absorptive cover, water saturated, and kept continuously wet. Cover concrete surfaces and edges with 12-inch lap over adjacent absorptive covers.

2. Moisture-Retaining-Cover Curing: Cover concrete surfaces with moisture-retaining cover for curing concrete, placed in widest practicable width, with sides and ends lapped at least 12 inches, and sealed by waterproof tape or adhesive. Cure for not less than seven days. Immediately repair any holes or tears during curing period using cover material and waterproof tape.
a. Use moisture-retaining cover curing for topping slabs. Take extreme precautions when placing on textured, patterned, or colored topping slabs. Topping slabs damaged as the result of improper curing processes are

subject to rejection, removal, and replacement by Contractor at no additional cost to the contract.

b. *Moisture cure or use moisture-retaining covers to cure concrete surfaces to receive floor coverings.*
c. *Moisture cure or use moisture-retaining covers to cure concrete surfaces to receive penetrating liquid floor treatments.*
d. *Cure concrete surfaces to receive floor coverings with either a moisture-retaining cover or a curing compound that the manufacturer certifies will not interfere with bonding of floor covering used on Project.*

3. *Curing Compound: Apply uniformly in continuous operation by power spray or roller according to manufacturer's written instructions. Recoat areas subjected to heavy windfall within three hours after initial application. Maintain continuity of coating and repair damage during curing period.*

E. <u>ACI 306 R.1 and Commentary Indicates:</u>

[Guide to Curing Concrete] section 1.2 "Definition of curing" defines "curing period" as *"the time period beginning at placing, through consolidation and finishing, and extending until the desired concrete properties have developed"*

Section 1.4.5 "Duration of Curing" notes *"…curing should be continued until the required concrete properties have developed or until there is a reasonable assurance that the desired concrete properties will be achieved after the curing measures have been terminated and the concrete is exposed to the natural environment."*

and

> *"…it is common to permit termination of curing measures when the compressive strength of the concrete has reached 70% of the specified strength. This is a reasonable practice if the anticipated postcuring conditions allow the concrete to continue to develop to 100% of the specified strength within the required time period. When postcuring conditions are not likely to allow the required further development of concrete properties, it may be more reasonable to require curing until the concrete has developed the full required properties."*

We note the more restrictive WMATA requirements regarding curing are not noted within the Contract Documents.

F. As-Designed Structure (the majority of slabs)

Attachment 27 indicates the results of the analysis of the randomly selected as-designed representative elements of the structure, which indicate they were designed inducing stresses beyond those allowed by the Code, WMATA requirements, and Industry Standards.

In summary:

- Slabs - design is adequate for shear tension, torsion, and flexure with 8,000 psi and 10-inch slabs, including tolerances, but exceeds allowed initial and service level stresses.
- Pour Strip slabs with post-tensioning – design is adequate, but exceeds initial stress and service level stresses.
- Pour Strip slabs without post-tensioning - not adequate for shear, tension, torsion, or flexure, and exceed allowed initial and service level stresses.
- Beams - insufficient as designed and detailed for shear and torsion for ±20% of the ±10% of the total beams analyzed, as well as exceed initial and service level stresses.
- Girders – insufficient as designed and detailed for shear and torsion for ±30% of the ±12% of the girders analyzed, as well as exceed initial and service level stresses.
- Column design - acceptable.

G. Comparison of Certain Contract Document Requirements vs. In Situ

Table 2. Entrained Air Content

	Contract Document Drawings	Contract Document Specifications	Approved Mix Design	As-Built Per RBB field reports	As-built per APS	As-built per RJ Lee	As-built per JTC	As-built per TEC	As-built per UCT
Criteria (Tolerance)	6% (±1%)	5.5% (±1%)	5.5% (-1.5%)	4%-6.5% but some as low as 2%	2.0%-4.3%	2.3%-5.5%	4%-9%	5.4%-8.2%	4.9%-10.2% (Cols. SCC 2.0%-4.9%)

Note: Mix 8K1DC4NL, which calls for one gallon of DCI per 27 cubic feet (1 cubic yard) was approved by PB and WMATA, though we don't know where it was used, as no record for its use in deck pours was found in RBB field reports.

Mix 8K2DC2NL, also approved by PB and WMATA, calls for 2 gallons of DCI per 27 cubic feet (1 cubic yard) (as approved with a water/cement ratio – 0.29) approved by PB and others and poured in the decks based on RBB reports of deck pours.

Table 3. Water-Cement Ratio

Contract Document Specifications	Approved Mix Design	Per RBB Compressive Strength Test Results	As-built per APS core tests "with 0.05 tolerance"	As-built per RJ Lee	As-built per APS	As-built per JTC	As-built per TEC	As-built per UCT
<0.40	0.29	0.24-0.26	0.38-0.45	0.30-0.35	0.38-0.44	0.38-0.45	0.35-0.41	0.40 ± 0.05

Table 4. Slab Concrete Strength

Contract Document Drawings (Sheet S1.00)	Approved Mix Design	As-Built per RBB tests cylinder breaks	Compressive Strengths as-built per APS Core Tests (psi)	Compressive Strengths as-built per JTC Core Tests (psi)	Compressive Strengths as-built per TEC Core Tests (psi)	Compressive Strengths as-built per UCT Core Tests (psi)
8,000 psi	12,360 psi*	>>8,000 psi	**3,850 Therefore 4250 9,550	5,940-11,620	6,200-8,650	4,000 - ±11,000

*Per test results of mix design submitted and approved based on historical data.
** We believe one core was taken through a cracked section and the 3,850 psi results should be discarded.

Table 5. Slab Thickness Considering ACI-117-90 Section 4.4.1 Dimensional Tolerance Slab Thickness
(Attachment 23)
(Attachment 24)
(Attachment 25)

Contract Document Drawings	Allowed ACI 117-90 Tolerance	Acceptable Range	In Situ As-Built
10" slabs	+3/8", -1/4"	10-3/8" – 9-3/4"	7"- 12-1/4"

Crowning, camber, etc. not included.

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Table 6. Minimum Concrete Cover For Mild Steel Reinforcement

Face of Element	ACI 318-02	Per Contract Documents	ACI 117-90 Tolerance	Code Required considering 1-hour fire rating requirement	Code Required considering 2-3-hour fire rating requirement (Permit basis)
Top	2” for #6 bars or greater	2”	-1/ 2”	n/a	n/a
Bottom	2” for #6 bars or greater	1”	-1/ 4”	3/4”	3/4”

Table 7. Minimum Concrete Cover for Post-Tensioning Conduit

Face of Element	ACI 318-02	As Designed	ACI 117-90 Tolerance	As Designed Including Tolerance	Code Required (after considering concrete protection & fire rating requirement)
Top	1-1/2” girders and beams, 1” slabs	2”	3/8” for duct	1-5/8”	n/a
Bottom	1-1/2” girders and beams, 1” slabs	2” – 2-1/2”	3/8” for duct	1-5/8” – 2-1/8	3/4”

Table 8. Allowable Stress in Post-Tensioned Concrete

	ACI 318-02	WMATA	Contract Document Design Calculations
Initial Stage	3√f’c	6√f’c	No information
Service Stage	7.5√f’c	Not noted	6√f’c
Ultimate Load	12√f’c**	Not noted	Not noted

** after “cracked” as defined by ACI.

VII. IN SITU TOPOGRAPHY AND THREE-DIMENSIONAL SURVEYS

After we reviewed the surveys performed by others, Rice performed both a topographic survey of the top of the slabs and a three-dimensional scan of Levels 330 and 350. The purpose of these surveys was to determine vertical elevations of the tops and bottoms of the elevated levels, their thickness, and where possible, dimensions of the projection beams, girders, and columns. After completing the initial topographic surveys, a more detailed three-dimensional survey was performed. Results of the three-dimensional survey yielded the in situ slab thickness, approximate beam widths and depths, girder widths and depths, and orthogonal column dimensions.

This three-dimensional survey performed indicated that the slab thickness varies from less than 7 to 12-1/4 inches where there are no cast-in-place curbs or walkways present (as those thicknesses could not be measured). Note that based on the minimum 10-inch slab thickness noted on the Contract Documents combined with the ACI 117-90 thickness tolerances (plus 3/8 inch, minus 1/4 inch), all slabs that are less than 9-3/4 inches or greater than 10-3/8 inches do not comply with ACI 117-90; however, the maximum slab thickness limits would not apply because crowned and cambered slab requirements would override the maximum thickness limitation.

We would note that based on Rice's survey of slab thickness, the approximate percentage of slab thickness in the drive aisles is as follows:

Level 330

- 3.5% less than 9"
- 18% greater than 9" and less than 9-3/4"
- 44 % greater than 9-3/4" and less than 10-3/8"
  Note: below includes the extra thickness required to accommodate crowning, camber, and sloping per Contract Documents
- 32% greater than 10-3/8"

Level 350

- 2.5% less than 9"
- 20% greater than 9" and less than 9-3/4"
- 38 % greater than 9-3/4" and less than 10-3/8"
- Note: below includes the extra thickness required to accommodate crowning, camber, and sloping per Contract Documents
- 38% greater than 10-3/8"

We received a survey previously performed by Greenhorn & O'Mara (Attachment 26) that included an illegible legend on two of the sheets in the copy we were provided, but it appears their thickness determinations in drive aisles may agree with the Rice surveys.

The Contract Document drawings specify, among other notations, "*Camber structure to provide secondary cross slopes to meet elevations noted*."

and

"…*concrete thicknesses noted are minimums*…"

The "camber" of the slab was discussed in the responses to RFI 216 and 571 indicating the "crown" was to be to be top only, as no mention was made of bottom. The bottoms of the slabs (soffits) therefore were, in certain cases, installed uncambered. This yields beyond "maximum" thickness slabs being installed.

VIII. FIELD INVESTIGATION

Our field investigations included visual observations, nondestructive testing, cutting of inspection openings, measurements, extraction of concrete cores, concrete materials testing, and grout materials testing.

A. General Visual Observations

Visual observations were collected in order to determine the location and extent of distress. The following summarizes our observations.

1. Typical Slabs

Each of the elevated slabs is specified to be minimum 10 inches thick and in general is graded (crowned) to drain water away from the drive aisles and to catch basins. The slabs are reinforced with a combination of grouted post-tensioning and epoxy coated mild steel reinforcing bars. Portions of each elevated slab are covered by cast-in-place concrete curbs and walkway overlays over waterproofing.

Visual observations were made of the tops and bottoms of the elevated slabs. The top-of-slab observations did not include areas that were covered by the concrete curbs and walkways. The bottom-of-slab ("soffit") observations were typically made from the level below. At selected locations scissor and aerial lifts were utilized to perform up-close soffit inspections.

The post-tensioned slabs evidenced partial depth cracking, full depth cracking, and thin cementitious patches. In addition, ponding water was noted at discrete locations on the Level 350 slab. Seven locations exhibited post-tensioned ducts exposed at the top surface of the slab, and at two openings through the pedestrian overlay (made by others). Numerous cracks were noted to have been epoxy injected or were being injected during our visits. Slab observations noted at the time of our initial visual observations for each elevated level are included in Attachment 28.

2. Beams and Girders

The floor beams are designed to be approximately 36 inches deep (including the 10-inch thick slab) by approximately 18 inches wide. The girders are designed to be approximately 72 inches deep (including the 10-inch thick slab) and approximately 36 inches wide. Note that the typical 72-inch girder depth dimension does not include the 24-inch taper that is present at many girder ends. Therefore, at the deepest point of the tapered section, girders are approximately 96 inches deep adjacent to the supporting

column. The typical beam and girder spans are approximately 40 feet and 80 feet, respectively.

Close-up visual surveys were performed at selected beam and girder soffits of Levels 330 and 350 using scissors and aerial lifts. Vertical and diagonal cracks were observed in numerous beams and girders. Crack widths ranged from hairline (less than or equal to 0.007 inches) to 0.030 inches.

Beam cracks tended to belong to one or two categories: the first group included primarily vertical cracks that appeared to be related to the restraint of volumetric changes. The second group was noted more near beam ends, where vertical cracks divided into multiple segments close to the beam soffit (bottom). This second crack pattern is consistent with post-tensioning forces induced at an early age. Similar to the slab cracks, numerous beam cracks were observed to have been epoxy injected.

At the girders, cracking patterns were typically observed near girder ends, with cracks branching out towards the soffits of the girders. Again, these cracks are consistent with post-tensioning forces induced at an early age. Attachment 29 photos illustrate typical beam and girder cracking.

3. Columns

The square and rectangular columns vary in size but typically have dimensions ranging from 44 to 62 inches. The circular columns range from 24 to 64 inches in diameter. Some of the columns are, by design, integrated into the perimeter concrete walls.

Close-up visual surveys were performed from the deck level at columns extending upward from the 305 and 330 Levels. Horizontal and diagonal cracks were observed at 46 columns (of approximately 78 columns observed). Cracks were generally more prominent on the outboard face of the column toward the main girder framing into that column (i.e., on the South face of B-line columns, the North face of C-line columns, etc.).

Another group of cracks in the columns was observed at an inspection opening that was related to void formation under the column ties during concrete placement.

Each of these styles of cracks was typically located above and within six feet of each elevated slab (frequently in the sandblast finished portion of the columns). Various forms of crack repair were observed, including epoxy injection and sand-cement parging. Crack widths at the columns typically ranged from hairline (less than or equal to 0.007 inches) to 0.020 inches. Attachment 30 is a photographic illustration of typical column cracking.

B. <u>Nondestructive Evaluations</u>

1. Ground Penetrating Radar (GPR) and Cover Surveys

GPR is a nondestructive test method that utilizes electromagnetic energy to characterize as-built construction of concrete elements. In this test a pulse of radio frequency

energy is emitted into the concrete. When the energy encounters a material with a different dielectric constant than what it "sees" in its initial "penetration," such as reinforcement, a portion of that energy is reflected back to the antenna. By measuring the time for that energy to be reflected, the depth of internal features can be determined.

We performed GPR and cover meter surveys at the slabs, beams, girders, and columns using a Geophysical Survey Systems, Inc. (GSSI) SIR-3000© unit with antennas ranging from 1.0 GHz to 2.6 GHz; a GSSI StructureScan Mini© (1.6 GHz antenna); and a GSSI StructuresScan Mini© HR (2.6 GHz antenna). In order to calibrate the depth of the tendons and reinforcing bars, we cut inspection openings (approximately 12 inches by 12 inches) in the concrete after drilling probe openings (1/2 inch diameter).

These openings allowed physical measurement of the post-tensioning tendons and reinforcing bar depths, and thus, calibration of the GPR equipment. We also used an Elcometer 331 Concrete Covermeter© to collect concrete cover measurements over reinforcing.

Attachment 31 illustrates the location of the slabs, beams, and girders where this nondestructive testing was performed, as well as where cores were taken.

a. <u>Typical Slabs</u>

GPR surveys were performed in representative slabs in 20 bays on Level 330 and 14 bays on Level 350. The size and distribution of the areas selected for surveying provided tendon information (i.e. profile and top-of-slab cover) for approximately 40 to 50 percent of the drive aisle surface area of each elevated slab.

This report and other previous reports and analyses reference "thin" and "thick" slabs. For this report, "thin" slabs are defined as slabs as called for on the Contract Documents that do not comply with the ACI 117-90 minimum tolerance (which results in a minimum permissible slab thickness of 9-3/4 inches). Similarly, "thick" slabs are defined as slabs that do not comply with the upper limit of ACI 117-90 tolerance, which results in a maximum permissible slab thickness of 10-3/8 inches; however, it should be noted, that the slab thickness shown on the Construction Documents are noted as the minimum and require having the top crowned to slope.

All of the GPR work was performed from the top surface of the elevated drive aisle slabs in order to collect tendon high and low point information. This work did not include areas below the curbs and walkways since those areas would have required scanning from the bottom of each slab and could not provide information about the tendon high points (due to limitations of the technology) at the slab-to-beam interface. The GPR survey focused on locating both uniformly-distributed as well as temperature and shrinkage tendons.

Attachment 32 illustrates the slab uniformly distributed tendon information that was collected. Notes relating to that attachment are as follows:

i. All of the referenced tendon profiles are taken from the VSL shop drawings. Those drawings provide support chair heights plus the offset value to the tendon centroid. In order to compare the tendon profiles with the GPR data, we plotted the top of the strands within the tendon, not the centroid of the tendon.

ii. Although continuous scans were collected, only average slab tendon profiles are presented for the irregularly shaped slab areas (i.e. wedge sections in the curved areas of the ellipse). These wedge-shaped areas do not allow us to measure average cover and profile data since individual tendon geometry varies across the slab area.

iii. While collecting the slab tendon profiles, one missing tendon was noted at Slab A-B/7-8 on Level 330. In addition, several slab areas were found to have tendons with less than 2 inches (modified to 1-5/8 inches by PB in selected locations) of concrete cover minus 3/8 inch tolerance). Attachment 33 provides top-of-slab concrete cover and tendon profile information.

b. Pour Strips

At Level 330, there are Pour Strips on the East and West ends of the structure. Each of the Level 330 Pour Strips is ±10 feet wide, net, North-South by approximately 76 feet long East-West. At Level 350, there is one Pour Strip on the East end of the structure (approximately 20 feet wide North-South by approximately 40 feet long East-West).

The purpose of the Pour Strips is to allow access to the adjacent post-tensioning anchors during stressing, as well as to behave as a temporary construction expansion area to separate adjacent concrete pours. This allows construction phase shortening and shrinkage to occur. These were “poured in” after adjacent pours were a minimum sixty days old and after the concrete had achieved full strength based on cylinder compressive strengths. The Pour Strips do not act as expansion joints in the completed Project.

It is our opinion that the Contract Document drawings indicate that the Level 330 Pour Strips were to be reinforced with mild steel and post-tensioning tendons. However, the VSL shop drawings, which were approved by PB, do not show post-tensioning tendons in these areas, nor did the RBB report indicate that. In addition, in its formwork approval of this area, PB indicated it was not post-tensioned.

Our GPR survey did not detect the presence of post-tensioning tendons within either of the Level 330 Pour Strips. In addition, at the West Pour Strip at Level 330, North-South temperature and shrinkage mild steel reinforcing was not

present. The lack of North-South temperature and shrinkage reinforcing followed the PB reinforcing steel shop drawing edits.

c. <u>Beams and Girders</u>

GPR surveys were performed at 42 beams and 5 girders on Level 330 and 20 beams and 7 girders on Level 350. The Attachment 34 photo illustrates a typical beam/girder survey while in process. The beams and girders surveyed represent approximately 20 and 25 percent, respectively, of the total number of beams and girders present in the structure under the drive aisles.

These survey locations were randomly selected to provide a representative sample of various structural conditions within each elevated level. These included single span girders; continuous girders and beams; girders and beams in the straight portions of the structure; and girders and beams in the curved portions of the structure.

All of the work that was performed focused on locating mild steel top and bottom reinforcing bars (where possible), stirrups, and the post-tensioning tendons. In some cases, the number of top reinforcing bars was not able to be determined due to the large quantity and therefore density of adjacent reinforcing, which concealed or "ghosted" the GPR. The number or spacing of reinforcing bars and the post-tensioning tendon position were used to verify general conformance with the Contract Document locations.

Due to limitations imposed by the physical size of the GPR units and available space, there were areas of the beams and girders that could not be surveyed. The GPR units utilized for this Project were relatively small so the areas that could not be scanned were where access was typically less than 6 inches wide. These areas are hatched on the elevations in the attachment and are typically located at each end of each member.

By design, many of the girders and beams have draped post-tensioning tendon profiles. For reference, draped tendons closely resemble a parabolic shape. The girder tendon arrangement typically changes from a vertical orientation (i.e. one column of three tendons) at the girder ends to a horizontal orientation towards mid-span (i.e. one row of three tendons). Within that draped profile, the relative placement of tendons typically also varies along the length. For example, near mid-span, the tendons may be arranged side-by-side in a horizontal fashion to (i.e., one row of three tendons), while near girder ends, the tendons are typically configured in a stacked arrangement (i.e., one column of three tendons).

At the girder ends, where the tendons are stacked vertically, the as-designed concrete side cover is approximately 16 inches. The side cover, or clear distance from the outermost vertical surface of the concrete to the face of the tendon group, made accurate GPR location challenging due to penetration limitations of the instrument. While some of these tendons could be located, others could

not, even when using a high-penetration 1.0 GHz antenna. Note that as the frequency of a GPR antenna decreases, the antenna's penetration power increases but the signal resolution decreases. Where the girder tendons could not be positively located with the 1.0 GHz antenna, no in situ tendon location data was recorded.

GPR results identified 60 beams with tendons that deviated from the as-designed tendon profile by more than the PB as-designed location with ACI placement tolerance incorporated. The maximum tendon deviation observed was at Beam 7 (PB93 - 350 Level, A-B/9-10). Attachment 35 illustrates the girder tendon deviation information while Attachment 35A illustrates the beam GPR results. Note that the as-designed tendon profiles were determined using the VSL shop drawings.

Each of the girder tendons surveyed had areas that deviated from the as-designed tendon profiles by more than the ACI 117-90 placement tolerance. The maximum tendon deviation observed was at Girder I (PG45 - 350 Level, B-C on Line 5). Attachment 36 illustrates a summary of the beam tendon deviation information. Note that the as-designed tendon profiles were determined using the VSL shop drawings.

d. Columns

At each of the girders observed, a GPR survey was performed at one or more faces of the supporting columns. This work focused on locating the vertical reinforcing bars and horizontal reinforcing (ties) in the columns. Concrete cover was estimated using an Elcometer 331 Covermeter©. Measured depths to the vertical bars and ties were compared to the Contract Document requirements (minimum cover 2 inches ± ACI 117-90 tolerances).

Column vertical reinforcing bar cover ranged from 0.3 inches to greater than 3 inches. The average of the cover in columns is 0.9 inches. Attachment 37 illustrates the column concrete cover dimension.

Column C7 above Levels 305 and above Level 330 appears to have two relocated bars on the North face of the column. We would note PB made numerous changes to column reinforcing locations to change the placement of the as-designed column vertical and lateral reinforcement to allow the placement of the as-designed deck post-tensioning (Figure 1), and this relocation was one of those changes.

Figure 1.

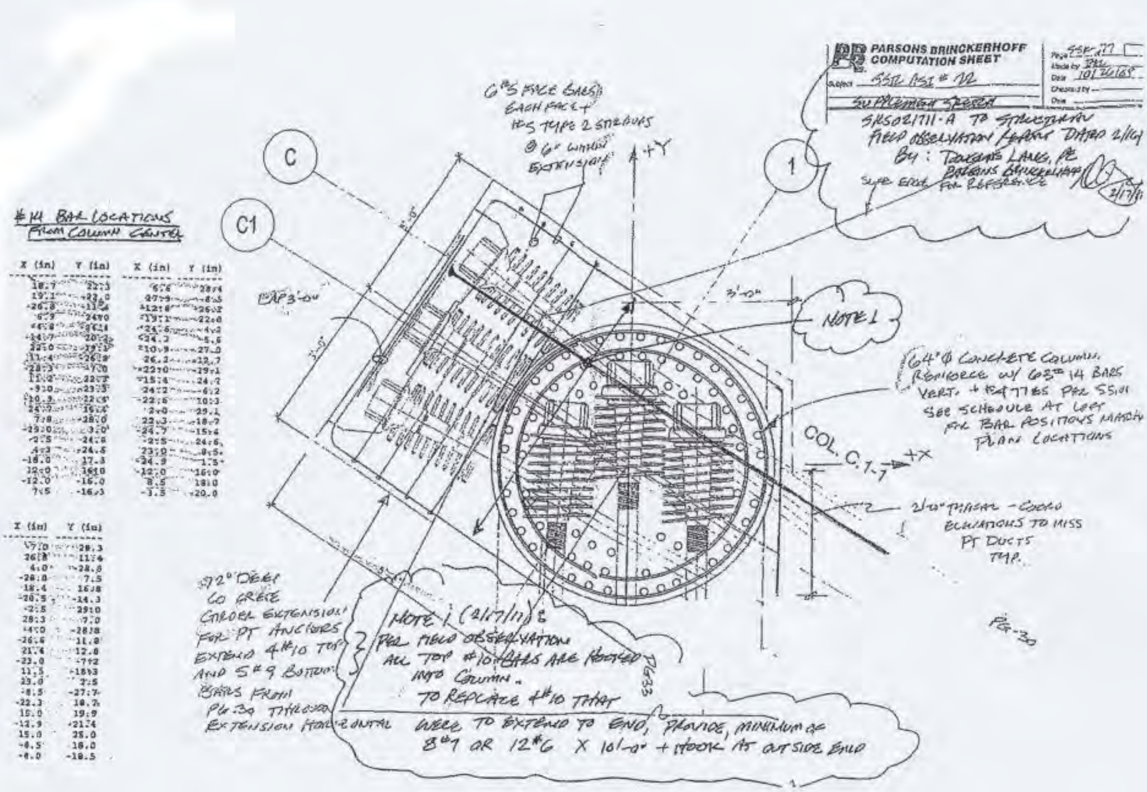

FIGURE 1

2. Impulse-Response Testing

Impulse-Response testing is a nondestructive method used to evaluate the as-built condition of the concrete qualitatively and to detect the presence or absence of delaminations in the concrete. The test consists of striking the concrete surface with a hammer with a load cell attached and measuring the impulse applied to the concrete element. At the same time, the overall velocity response of the element is measured using a Geophone. With this information, a mobility plot is developed for a given test location. When performed over a grid of points, qualitative information about the presence or absence of delaminations can be detected. In general, locations that have internal delaminations exhibit higher average mobility values than locations that do not have internal delaminations.

Impulse-Response testing was performed at three representative locations on Levels 330 and 350. These representative locations were selected based on the presence of top surface cracks, and/or exposed post-tensioning in order to determine if delaminations in the deck that were not visible at the surface had occurred. Impulse-response test locations are summarized in Attachment 38.

Impulse-Response testing did not detect areas with significantly higher average mobility values, and therefore, suggests no internal delaminations were present in the areas tested. Results of impulse response testing are summarized in Attachment 39.

3. Impact-Echo Testing

We visited the site immediately after short duration rains and observed as the water evaporated, leaving water stains (which then evaporated) in cracks that had the rainwater fall or run to them, which led us to separately perform Impact-Echo testing in various locations to determine the width and depths of other almost visually non-discernible cracks.

The Impact-Echo test method is a non-destructive procedure that utilizes low-strain stress waves to characterize internal features or distress in concrete elements. The test consists of imparting a stress wave into the concrete by impacting it with a steel sphere and measuring the propagation of the wave through the concrete. When the time for the wave to propagate around the crack is measured, the depth of cracks can be estimated.

Impact-Echo was used to estimate the depth of surface-opening cracks that were visible at the top surface of the drive aisle slabs.

The locations for Impact-Echo testing and results are summarized in Attachment 40. Testing focused on cracks at the top surface since cracks such as these will have the most significant effect on the long-term durability of the structure. At the time of initial observations, this cracking was concentrated in the southern half of Level 330 and on the Eastern portion of Level 350 (Attachment 41). Cracks that were tested on these

levels exhibited widths that varied between 1/200th of an inch and 1/64th of an inch. A total of 35 crack locations were tested on Level 330, and 13 locations on Level 350.

Crack depths were estimated to extend between 1/2 of an inch down from the top of the slab and through the full depth of the slab at the locations tested. The average crack depth is estimated at 5 inches with a standard deviation of 2.1 inches.

IX. DESTRUCTIVE TESTING

A. Inspection Openings

Attachment 42 summarizes general location of our beam, girder, slab, and column inspection openings and our materials testing locations. The photographs shown in Attachment 43 illustrate inspection openings in a slab, girder, and column, respectively.

We cut inspection openings to observe the condition of the post-tensioning sheathing and tendons, to calibrate the GPR equipment, and to confirm our GPR scan data. The typical size of an individual inspection opening was 12 inches x 12 inches. The depth of each inspection opening varied based on the depth of the reinforcing bar or tendon found at the opening under consideration. The inspection openings were selected to provide a representative physical sample of the various structural elements and conditions at each elevated level.

We exposed post-tensioning tendons in 36 of the 49 inspection openings and grout samples were obtained from 27 of those locations. It is our understanding that Whitlock Dalrymple Poston & Associates, Inc. (WDP), who was retained by WMATA, obtained grout samples at each of the 9 inspection openings. Note that we did not obtain grout samples from the WDP inspection openings.

After we collected grout samples from inside the duct and documented general conditions (including concrete cover dimension and grout and strand condition), we replaced the grout and repaired both the duct and opening. (During the course of our investigation, one wire (within a seven wire strand) was damaged.)

The opening repairs were performed in general accordance with a repair procedure that was submitted to and approved by Montgomery County and WMATA. We have deemed the repair of the one wire we damaged as not necessary.

Inspection opening observations included one minor slab tendon grout void (Attachment 44) at Slab Opening 1 (Level 330 A-B/7-8).

Concrete cover at nine of the locations measured was less than the requirements of the Contract Documents (including the appropriate ACI 117-90 cover tolerance).

B. Cores

Prior to the extraction of each core, we performed GPR surveys from the top and bottom of the elevated slabs. After accidentally damaging one mild reinforcing bar at Core 10, we retained

Testing Technologies, Inc. (TTI) to perform radiographic testing in accordance with County, State, and Federal regulations. Thereafter, each core location was first scanned using GPR and then radiographically tested. Both activities were performed to limit or avoid damage to the embedded post-tensioning tendons, reinforcing bars, and electrical conduits. Only one reinforcing bar was damaged during our initial ROM coring activities and six during the subsequent secondary coring.

The radiograph evaluations were limited by the radiation source and the clear distance standoff required. The radiographic testing method consisted of exposing reactive film to a radiation source. The results of this testing generally confirmed the GPR work by locating the embedded items previously noted above. At Core 10, one bottom mat reinforcing bar was damaged during coring. (Repair procedures have been sent under separate cover, as well as responses to RBB tests run on those repair materials).

In our rough order of magnitude (ROM) initial sampling, a total of thirty-four 4-inch nominal diameter cores were extracted from the elevated slabs, twenty-one at Level 330 and thirteen at Level 350 (Attachment 45). Twenty-two of the cores were shipped to APS for ROM compressive strength testing and full petrographic examination while twelve cores were shipped to RJ Lee for material testing to calibrate the service life model. Subsequently, Core 11 was forwarded from APS to RJ Lee. No initial testing was performed on Core 10, as that core contained a reinforcing bar.

After the review of the initial ROM compressive strength data suggested that the in situ concrete strength may be lower than the design value, it was decided that this core could still provide usable strength information if the bottom of the core containing the reinforcing bar was cut off. Subsequently, compressive strength testing was performed by APS after core preparation.

The APS ROM and supplemental concrete materials testing results data suggested that the average compressive strength was below the design strength, including several test results below 6,000 psi. Therefore, it was decided that a more rigorous assessment of concrete strength should be performed and a total of 78 additional cores were extracted, a minimum of three from each slab (NB: no additional cores were taken from Pour Strips due to their inherent unacceptability due to cracking and missing reinforcing), and several from columns. Again, GPR and radiographic testing were performed to locate embedded reinforcement and other internal features prior to coring. These cores were shipped to three different materials testing laboratories (TEC, UCT, and JTC) for compressive strength testing and petrographic studies. Table 9 presents a listing of cores taken, with location, core number, which lab tested, and tests performed. Note that the core designation includes a description for the area sampled (such as concrete pour 1A, Column B/10, etc.), the core number, the agency performing testing, and a description of the test performed. In total, approximately five reinforcing bars were damaged (nicked or cut) during the secondary coring operations since they were beyond the depth penetration of the X-ray and GPR.

Table 9. Core locations and other information

Core	Core	Core	Core
WPS - 1 - APS - Comp	1C - 44 - JTC - Comp	1G - 85 - UCT - Comp	2B - 126 - UCT - Comp
WPS - 1A - RJL - Dura[1]	1C - 45 - UCT - Comp	1G - 86 - UCT - Petro	2C - 127 - JTC - Petro
WPS - 2 - APS - Comp	1C - 46 - UCT - Comp	1H - 87 - JTC - Comp	2C - 128 - JTC - Comp
WPS - 3 - RJL – Petro	1C - 47 - JTC - Petro	1H - 88 - JTC - Comp	2C - 129 - JTC - Comp
WPS - 4 - RJL - Dura[1]	1C - 48 - UCT - Petro	1H - 89 - UCT - Comp	2C - 130 - JTC - Comp
WPS - 4A - APS - Comp	1C - 49 - UCT - Hold [3]	1H - 90 - JTC - Comp	2C - 131 - UCT - Petro
WPS - 5 - RJL – Dura	1C - 50 - JTC - Comp	1H - 91 - UCT - Comp	2C - 132 - UCT - Comp
1C - 6 - APS – Comp	1A - 51 - UCT - Comp	1H - 92 - UCT - Comp	2C - 133 - UCT - Comp
1C - 7 - APS – Petro	1A - 52 - UCT - Comp	1H - 93 - JTC - Petro	2C - 134 - UCT - Comp
1C - 8 - APS – Comp	1A - 53 - JTC - Comp	1H - 94 - UCT - Petro	2D - 135 - JTC - Petro
1C - 9 - APS – Comp	1A - 54 - WJE - Comp	1E - 95 - JTC - Comp	2D - 136 - JTC - Comp
1C - 10 - APS – Comp	1A - 55 - UCT - Comp	1E - 96 - JTC - Petro	2D - 137 - JTC - Comp
1F - 11 - RJL – Petro	1A - 56 - TEC - Petro	1E - 97 - JTC - Comp	2D - 138 - JTC - Comp
1F - 12 - APS – Comp	1A - 57 - UCT - Petro	1E - 98 - JTC - Comp	2D - 139 - UCT - Comp
1F - 13 - APS – Comp	1A - 58 - JTC - Petro	1E - 99 - UCT - Petro	2D - 140 - UCT - Comp
1F - 14 - APS – Comp	1A - 59 - JTC - Comp	1E - 100 - UCT - Comp	2D - 141 - UCT - Petro
1F - 15 - RJL – Dura	1A - 60 - TEC - Hold [2]	1E - 101 -UCT - Comp	2D - 142 - UCT - Comp
EPS - 16 - APS - Comp	1A - 61 - TEC - Hold [2]	1E - 102 - UCT - Comp	B/2/330 - 143 - UCT - Petro
EPS - 17 - APS - Petro	1A - 62 - TEC - Hold [2]	1F - 103 - JTC - Comp	B/2/330 - 144 - UCT - Comp
EPS - 18 - APS - Comp	1B - 63 - JTC - Comp	1F - 104 - JTC - Comp	B/2/330 - 145 - UCT - Comp
EPS - 19 - APS - Comp	1B - 64 - UCT - Comp	1F - 105 - JTC - Petro	B/2/330 - 146 - UCT - Comp
Note: Cores 20-25 were not extracted.	1B - 65 - UCT - Comp	1F - 106 - JTC - Comp	B/10/330 - 147 - JTC - Petro
	1B - 66 - JTC - Comp	1F - 107 - UCT - Comp	B/10/330 - 148 - JTC - Comp
2A - 26 - APS – Comp	1B - 67 - JTC - Comp	1F - 108 - UCT - Petro	B/10/330 - 149 - JTC - Comp
2A - 27 - RJL - Dura[1]	1B - 68 - UCT - Comp	1F - 109 - UCT - Comp	B/10/330 - 150 - JTC - Comp
2A - 28 - APS – Comp	1B - 69 - UCT - Petro	1F - 110 - UCT - Comp	C/7/305 - 151 - UCT - Petro
2A - 29 - RJL – Petro	1B - 70 - JTC - Petro	2A - 111 - JTC - Comp	C/7/305 - 152 - UCT - Comp
2A - 30 - APS – Comp	1D - 71 - JTC - Petro	2A - 112 - JTC - Petro	C/7/305 - 153 - UCT - Comp
2B - 31 - APS – Petro	1D - 72 - JTC - Comp	2A - 113 - JTC - Comp	C/7/305 - 154 - UCT - Comp
1F - 32 - RJL - Dura[1]	1D - 73 - JTC - Comp	2A - 114 - JTC - Comp	1C - 155 - UCT - Comp
1F - 33 - RJL - Dura[1]	1D - 74 - JTC - Comp	2A - 115 - UCT - Comp	1C-156 - TEC - Comp
2B - 34 - APS – Comp	1D - 75 - UCT - Comp	2A - 116 - UCT - Petro	1C-157 - TEC - Comp
2B - 35 - APS – Comp	1D - 76 - UCT - Comp	2A - 117 - UCT - Comp	1C-158 - TEC - Comp
2B - 36 - APS – Comp	1D - 77 - UCT - Petro	2A - 118 - UCT - Comp	1A - 159 - TEC - Comp
2A - 37 - RJL - Dura[1]	1D - 78 - UCT - Comp	2B - 119 - JTC - Comp	1A - 160 - TEC - Comp
2A - 38 - RJL – Dura	1G - 79 - JTC - Petro	2B - 120 - JTC - Comp	1A - 161 - TEC - Comp
1C - 39 - TEC – Petro	1G - 80 - JTC - Comp	2B - 121 - JTC - Comp	
1C - 40 - TEC - Hold [2]	1G - 81 - JTC - Comp	2B - 122 - JTC - Petro	
1C - 41 - TEC - Hold [2]	1G - 82 - JTC - Comp	2B - 123 - UCT - Petro	
1C - 42 - TEC - Hold [2]	1G - 83 - UCT - Comp	2B - 124 - UCT - Comp	
1C - 43 - JTC – Comp	1G - 84 - UCT - Comp	2B - 125 - UCT- Comp	

Notes:
1. Core samples were not tested.
2. Core samples prepped for petrography but not tested.
3. Extra 3-inch diameter core not tested

Attachment 46 summarizes the testing extraction and testing protocol for those cores.

GPR and radiographic testing were used to locate embedded items before the secondary cores were excised.

Cores in columns were taken after probing columns with a rotating star drill (hammer), which does not cut reinforcing, to locate areas where core samples could be made without cutting main reinforcing steel. One lateral tie approximately 5” deep was nicked with the core drill barrel during this process.

X. MATERIALS TESTING

A. Concrete Cores

APS initially performed petrographic analysis and ROM compressive strength testing on cores, well as grout materials testing (Attachment 47). Note that RJ Lee also performed concrete materials testing (not compressive strength) on cores; their work is discussed in the Durability Analysis section of this report.

1. Concrete Testing Results

We initially sampled the concrete slabs in several locations to determine a ROM of concrete material strength and forwarded them to APS for evaluation and petrographic examination.

a. APS initially performed petrographic examination, air content analysis, and chloride analysis on three cores.

Compressive strength testing was performed as noted. Note that since this was intended to be ROM testing, the core sampling process and compressive strength testing were not performed in strict accordance with ASTM C42 requirements. Therefore, these cores have not been used to accept or reject SSTC concrete or to determine in-place strength.

Note: cores were initially extracted from randomly selected representative samples of the deck, though cores were not excised from each pour. (Cores 20-25 were not removed).

A summary of the APS ROM initial concrete compressive strength testing results is as follows:

Compressive tests results ranged from 3,850* to 9,550 psi

i. Level 330 (3 cores per pour)
   a) Pour 1C - Max.= 6,690 psi, Min.= 4,620 psi, Avg. = 5,873 psi
   b) Pour 1F - Max.= 7,640 psi, Min.= 7,130 psi, Avg. = 7,313 psi
   c) East Pour Strip (EPS) – Max. = 8,350 psi, Min. = 3,850 psi, Avg. = 5,980 psi
   d) West Pour Strip (WPS) – Max. = 9,550 psi, Min. = 4,330 psi, Avg. = 6,446 psi

ii. Level 350 (3 cores per pours noted)
   a) Pour 2A - Max.= 8,660 psi, Min.= 5,620 psi, Avg. = 6,673 psi
   b) Pour 2B - Max.= 8,790 psi, Min.= 7,850 psi, Avg. = 8,170 psi

* We believe one core was taken through a cracked section and the 3,850 psi result should be discarded.

b. <u>APS "Follow-up" Testing</u>

Since the initial ROM test results were extremely variable, APS was asked to perform additional limited petrographic examination and compressive strength tests on samples they had in their possession.

Additional compressive strength testing of ROM sample cores was made on four cores (supplemental). Three of the cores that had been tested previously had a portion of the core that appeared to be undamaged. The following summarizes the compressive strength testing results:

i. Core 10 - 7,030 psi (not previously tested) (Attachment 48)

Attachment 48A:

ii. Core 26 - bottom portion of petrographic sample - 6,330 psi (compared to 8,660 psi from the original ROM testing)
iii. Core 28 - bottom portion of petrographic sample - 6,430 psi (compared to 5,740 psi from the original ROM testing)
iv. Core 30 - bottom portion of petrographic sample - 5,290 psi (compared to 5,620 psi from the original ROM testing

Petrographic examination of the initially evaluated ROM cores did not identify unique characteristics that would invalidate the initial compressive strength test results.

<u>Compressive Strength Testing for Acceptance Summary per ACI 318-02 (not including APS)</u>

See Tables 10, 10A, and 11.

75% f'c = 6000 psi
85% f'c average = 6800 psi

ACI 318, Chapter 5

"…5.6.5.4 – Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f'c and if no single core is less than 75 percent of f'c. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted…"

"…5.6.5.5 – If criteria of 5.6.5.4 are not met and if the structural adequacy remains in doubt, the responsible authority shall be permitted to order a strength evaluation in accordance with Chapter 20 for the questionable portion of the structure, or take other appropriate action..."

DPS has further advised they are the "responsible authority" and since the building is not complete and there has been no Certificate of Occupancy issued, Chapter 20 cannot be utilized for evaluation.

DPS has further advised it would allow, after proper consideration, lower strength concrete than the minimum specified on the Permit set if the Structural Engineer of Record submits proper documentation and accepts the lower strength concrete.

Table 10. Secondary Compressive Strength Core Results Compilation per Three Set Cores per Testing Load (Not Including APS) (> 0.75 = 6,000 psi; average > 0.85 = 6,800 psi)

Red indicates unacceptable concrete

	JTC		TEC		UCT		
	Break Results (psi)	**Average Break (psi)**	**Break Results (psi)**	**Average Break (psi) if 3 cores**	**Break Results (psi)**	**Average Break (psi) if 3 cores**	**Accepted/Not Accepted Per ACI 318-02**
Level 330							
1A	6,580 6,680 6,450	6,570	6,790 7,310 6,200	6,766	7,140 6,710 5,600	6,483	N
1B	9,320 8,950 5,490	7,920		—	8,560 6,720 6,470	7,520	N
1C	7140 6,080 9,440	7,553	8,110 8,650 8,230	8,330	7,610 6,760 6,560		Y
1D	7,100 9,300 9,480	8,626	—	—	9,780 9,980 6,280	8,680	Y
			—				
1E	9,370 9,490 9,440	9,416	—	—	10,700 8,780 5,070	8,183	N
1Ea		—	—	—		—	—
1Eb		—	—			—	—
1F	10,370 10,650 9,350	10,103	—	—	9,000 6,320 7,880	7,733	Y
1G	7,990 8,420 9,890	8,766	—	—	6,210 7,880 7,700		Y
1H	8,290 8,280 9,560	8,710	—	—	4,210 8,100 7,910	6500	N
Pour Strip East	8,350 8,740	—	—	—			
Pour Strip West	9,550	—	—	—			

Table 10 (cont.) Secondary Compressive Joint Secondary Strength Core Results Compilation per Three Set Cores per Testing Load (> 0.75 = 6,000 psi; average > 0.85 = 6.800 psi)

Red indicates unacceptable concrete

	JTC		TEC		UCT		
	Break Results (psi)	**Average Break (psi)**	**Break Results (psi)**	**Average Break (psi) if 3 cores**	**Break Results (psi)**	**Average Break (psi) if 3 cores**	**Accepted/ Not Accepted Per ACI 318-02**
Level 350							
2A	7,920 7,440 9,720	8,360	—	—	8,160 6,030 10,200	8,130	Y
2B	10,520 9,650 11,040	10,403	—	—	10,060 10,640 11,140	10,610	Y
2C	10,710 10,480 7,340	9,410	—	—	10760 5,330 10,460		N
2D	10,170 8,670 10,250	9,700	—	—	10,530 11,250 8,460	10,080	Y
Pour Strip East			—	—			
Col. B2 SSC			—	—	8,300 11,120 11,280	10,233	Y
Col. B10 SSC	11,180 11,620 11,200	11,333	—	—			
Col. C7 SSC			—	—	10,870 11,630 10,300	10,993	Y

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Table 10A. ACI 318-02 Compressive Strength Analysis (Not Including APS) - ASTM C42
Red indicates not acceptable per ACI 318

Location	Low (psi) Average (psi)	ACI 318-02 (1) Greater than 6,000 psi Greater than 6,800 psi
1A	**5,600** **6,540**	**No** **No**
1B	**5,490** 7,590	**No** Yes
1C	6,080 7,620	Yes Yes
1D	6,280 8,660	Yes Yes
1E	**5,070** 8,810	**No** Yes
1F	6,320 8,930	Yes Yes
1G	6,510 8,080	Yes Yes
1H	**4,210** 7,610	**No** Yes
2A	6,030 8,250	Yes Yes
2B	9,650 10,600	Yes Yes
2C	**5,330** 9,180	**No** Yes
2D	8,460 9,900	Yes Yes
Columns	8,300 10,830	Yes Yes

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Table 11. Concrete Compressive Strength Results per Pour (Not Including APS) - ASTM C 42

Core No.	Corrected Compressive Strength (psi)	Unit Weight (pounds/cubic foot)	Low (psi) High (psi) Average (psi) Notes
1A - 51 - UCT - Comp	7,140	146	5,600 7,310 6,540
1A - 52 - UCT - Comp	6,710	144	
1A - 53 - JTC - Comp	6,580	145	
1A - 54 - JTC - Comp	6,680	143	
1A - 55 - UCT - Comp	5,600	143	
1A - 59 - JTC - Comp	6,450	145	
1A - 159 - TEC - Comp	6,790	148	
1A - 160 - TEC - Comp	7,310	144	
1A - 161 - TEC - Comp	6,200	150	
1B - 63 - JTC - Comp	9,320	152	5,490 9,320 7,590
1B - 64 - UCT - Comp	8,560	143	
1B - 65 - UCT - Comp	6,470	139	
1B - 66 - JTC - Comp	8,950	151	
1B - 67 - JTC - Comp	5,490	141	
1B - 68 - UCT - Comp	6,720	144	
1C - 43 - JTC - Comp	7,140	146	6,080 9,440 7,620
1C - 44 - JTC - Comp	6,080	145	
1C - 45 - UCT - Comp	7,610	145	
1C - 46 - UCT - Comp	6,760	149	
1C - 50 - JTC - Comp	9,440	147	
1C - 155 - UCT - Comp	6,560	147	
1C-156 - TEC - Comp	8,110	147	
1C-157 - TEC - Comp	8,650	148	
1C-158 - TEC - Comp	8,230	147	
1D - 72 - JTC - Comp	7,100	144	6,280 9,980 8,660
1D - 73 - JTC - Comp	9,300	145	
1D - 74 - JTC - Comp	9,480	146	
1D - 75 - UCT - Comp	9,790	147	
1D - 76 - UCT - Comp	9,980	150	
1D - 78 - UCT - Comp	6,280	146	
1E - 95 - JTC - Comp	9,370	144	5,070 10,700 8,810
1E - 97 - JTC - Comp	9,490	149	
1E - 98 - JTC - Comp	9,440	147	
1E - 100 - UCT - Comp	5,070	149	
1E - 101 - UCT - Comp	10,700	146	
1E - 102 - UCT - Comp	8,780	146	
1F - 103 - JTC - Comp	10,370	145	6,320 10,650 8,930
1F - 104 - JTC - Comp	10,650	148	
1F - 106 - JTC - Comp	9,350	143	
1F - 107 - UCT - Comp	9,000	144	
1F - 109 - UCT - Comp	6,320	143	
1F - 110 - UCT - Comp	7,880	144	
1G - 80 - JTC - Comp	7,990	144	6,510 9,890 8,080
1G - 81 - JTC - Comp	8,420	141	
1G - 82 - JTC - Comp	9,890	144	
1G - 83 - UCT - Comp	6,510	143	
1G - 84 - UCT - Comp	7,880	149	
1G - 85 - UCT - Comp	7,770	141	

Table 11 (cont). Concrete Compressive Strength Results Per Pour (Not Including APS) - ASTM C42

Core No.	Corrected Compressive Strength (psi)	Unit Weight (pounds/cubic foot)	Low (psi) High (psi) Average (psi) Notes
1H - 87 - JTC - Comp	8,290	140	4,210 9,560 7,610
1H - 88 - JTC - Comp	8,280	139	
1H - 89 - UCT - Comp	4,210	130	
1H - 90 - JTC - Comp	9,560	141	
1H - 91 - UCT - Comp	8,100	145	
1H - 92 - UCT - Comp	7,190	148	
2A - 111 - JTC - Comp	7,920	146	6,030 10,200 8,250
2A - 113 - JTC - Comp	7,440	146	
2A - 114 - JTC - Comp	9,720	149	
2A - 115 - UCT - Comp	8,160	145	
2A - 117 - UCT - Comp	6,030	145	
2A - 118 - UCT - Comp	10,200	146	
2B - 119 - JTC - Comp	10,520	141	9,650 11,140 10,600
2B - 120 - JTC - Comp	9,650	145	
2B - 121 - JTC - Comp	11,040	147	
2B - 124 - UCT - Comp	10,060	145	
2B - 125 - UCT - Comp	10,640	152	
2B - 126 - UCT - Comp	11,140	150	
2C - 128 - JTC - Comp	10,710	145	5,330 10,760 9,180
2C - 129 - JTC - Comp	10,480	147	
2C - 130 - JTC - Comp	7,340	144	
2C - 132 - UCT - Comp	5,330	143	
2C - 133 - UCT - Comp	10,760	147	
2C - 134 - UCT - Comp	10,460	147	
2D - 136 - JTC - Comp	10,170	145	8,460 11,290 9,900
2D - 137 - JTC - Comp	8,670	142	
2D - 138 - JTC - Comp	10,250	143	
2D - 139 - UCT - Comp	10,530	144	
2D - 140 - UCT - Comp	11,290	147	
2D - 142 - UCT - Comp	8,460	142	
B/2 - 144 - UCT - Comp	8,300	142	8,300 11,280 10,230
B/2 - 145 - UCT - Comp	11,120	147	
B/2 - 146 - UCT - Comp	11,280	146	
B/10 - 148 - JTC - Comp	11,180	148	11,180 11,620 11,330
B/10 - 149 - JTC - Comp	11,620	149	
B/10 - 150 - JTC - Comp	11,200	149	
C/7 - 152 - UCT - Comp	10,870	148	10,300 11,630 10,930
C/7 - 153 - UCT - Comp	11,630	147	
C/7 - 154 - UCT - Comp	10,300	149	

Notes:

1. Cores numbers include the pour number or location, core number, company that performed the testing, and the type of test that was performed on the core.

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c. Compressive Strength Testing Summary

Tables 12A, 12B, and 12C summarize the secondary concrete compressive strength testing results (ASTM C42). These tables indicate the secondary test results only and do not include the initial ROM testing by APS.

The ASTM C42 data with the ACI 318-02 acceptance criteria is presented below.

d. Windsor Probe Testing

Due to the variations in the ROM concrete strength test results, we next elected to perform Windsor probe testing prior to extracting additional cores. Windsor probe testing relates hardness or penetration resistance to concrete compressive strength. The Windsor probe testing was performed adjacent to each of the original core locations. This work included firing three steel nails at each location into the concrete, within a small device, measuring the penetration, determining a Mohs scale of hardness, using manufacturer tables to estimate the concrete compressive strength, and averaging the three values at each location. Attachment 49 summarizes the Windsor probe results.

e. Secondary Testing

Based on our ROM concrete materials testing results (compressive strength and Windsor probe testing), we determined additional concrete materials testing was necessary. An additional 78 cores (a minimum of three per pour) were extracted, with no additional cores from Pour Strips. The sampling procedure used was per ASTM C42 and compressive strength testing in accordance with ASTM 39. In addition, we reviewed the broken portions of APS ROM tested cores. TEC performed both petrographic analysis and compressive strength testing (Attachments 50 and 50A). UCT performed compressive strength testing and petrographic examinations (Attachments 51 and 51A). JTC (aka WJE) performed compressive strength testing and petrographic examinations of the additional cores, as well as petrographic analysis and visual inspection of previously broken APS core and sections remaining (Attachments 52 and 52A).

We also performed a statistical analysis allowed by ACI, including all secondary tests, using ACI 214.4R-10, which provides guidance for determining the in situ concrete strength.

ACI 214.4R-10 is a statistical evaluation. We note that the standard deviation results are high in the secondary core strength results. Since the sampling set for the ROM test results included a substantially smaller number of tests, the concrete had been considered to be of what now is lower strength than when using a determination based on the 78 sample set, analyzed as allowed by ACI 214.4R-10.

The statistical work has been accomplished with the parameters set by ACI 214.4R-10. We selected the “alternative method” of analysis due to fewer small

sample size penalties. When developing the remediation procedure design, the results per pour via 214.4R-10 as shown in Tables 13A-13L must be kept in mind.

The following tables indicate the secondary concrete strength core breaks.

2. Compressive Strength Testing Evaluation Summary per ACI 214.4R-10 by Pour

Table 12A. ACI 214.4R-10 Slab Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	8,982	1,834	407	78	5,860	1.7	-	-	-
			90%	8,982	1,834	407	78	**6,050**	1.6	-	-	-
			75%	8,982	1,834	407	78	6,340	1.44	-	-	-
Modified Conventional	9-7	10%	95%	8,982	1,834	407	78	5,790	1.7	1.64	-	-
			90%	8,982	1,834	407	78	**6,000**	1.6	1.28	-	-
			75%	8,982	1,834	407	78	6,330	1.44	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	8,982	1,834	407	78	6,830	-	1.64	1.70	0.83
			90%	8,982	1,834	407	78	**6,970**	-	1.28	1.32	0.83
			75%	8,982	1,834	407	78	7,200	-	0.67	0.68	0.83

Table 12B. ACI 214.4R-10 Column Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	11,644	1,075	449	9	8,990	2.47	-	-	-
			90%	11,644	1,075	449	9	**9,340**	2.14	-	-	-
			75%	11,644	1,075	449	9	9,810	1.71	-	-	-
Modified Conventional	9-7	10%	95%	11,644	1,075	449	9	8,890	2.47	1.64	-	-
			90%	11,644	1,075	449	9	**9,270**	2.14	1.28	-	-
			75%	11,644	1,075	449	9	9,780	1.71	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	11,644	1,075	449	9	8,840	-	1.64	1.87	0.83
			90%	11,644	1,075	449	9	**9,030**	-	1.28	1.39	0.83
			75%	11,644	1,075	449	9	9,340	-	0.67	0.71	0.83

We have highlighted the 10 percent fractile and 90 percent confidence level values for each calculation method that was used (conventional, modified conventional, and alternative approach). In our engineering opinion, the alternative approach and its results are the most appropriate value for the concrete strength for this facility (i.e., f’c = 6,970 psi).

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Table 12.C. ACI 214.4R-10 Method Comparison By Pour

Pour Number	Conventional (psi)	Modified (psi)	Alternative (psi)
Level 330			
Pour 1A	5,870	5,820	5,500
Pour 1B	3,980	3,960	5,840
Pour 1C	5,710	5,680	6,210
Pour 1D	5,070	5,040	6,780
Pour 1E	4,300	4,280	6,740
Pour 1F	5,330	5,300	6,990
Pour 1G	5,870	5,820	6,490
Pour 1H	3,410	3,390	5,780
Level 350			
Pour 2A	4,760	4740	6,440
Pour 2B	9,810	9,700	8,810
Pour 2C	3,680	3,660	6,870
Pour 2D	7,570	7,530	8,070

It is our professional opinion, based on structural analysis, the concrete for all pours of decks on SSTC is 6,970 psi.

Tables 13A-13L indicate by pour the ACI 214.4R-10 results.

Table 13A. ACI 214.4R-10 Slab 1A Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5) K	Z	T	C
Conventional	9-6	10%	95%	7,091	569	295	9	5,690	2.47	-	-	-
			90%	7,091	569	295	9	**5,870**	2.14	-	-	-
			75%	7,091	569	295	9	6,120	1.71	-	-	-
Modified Conventional	9-7	10%	95%	7,091	569	295	9	6,000	1.71	1.64	-	-
			90%	7,091	569	295	9	**5,820**	2.14	1.28	-	-
			75%	7,091	569	295	9	5,670	2.47	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	7,091	569	295	9	5,390	-	1.64	1.87	0.83
			90%	7,091	569	295	9	**5,500**	-	1.28	1.39	0.83
			75%	7,091	569	295	9	5,690	-	0.67	0.71	0.83

Table 13B. ACI 214.4R-10 Slab 1B Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5) K	Z	T	C
Conventional	9-6	10%	95%	8,125	1,666	330	6	3,110	3.01	-	-	-
			90%	8,125	1,666	330	6	**3,980**	2.49	-	-	-
			75%	8,125	1,666	330	6	5,030	1.86	-	-	-
Modified Conventional	9-7	10%	95%	8,125	1,666	330	6	3,080	3.01	1.64	-	-
			90%	8,125	1,666	330	6	**3,960**	2.49	1.28	-	-
			75%	8,125	1,666	330	6	5,020	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	8,125	1,666	330	6	5,520	-	1.64	2.02	0.83
			90%	8,125	1,666	330	6	**5,840**	-	1.28	1.48	0.83
			75%	8,125	1,666	330	6	6,290	-	0.67	0.73	0.83

Table 13C. ACI 214.4R-10 Slab 1C Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	8,156	1,142	316	9	5,330	2.47	-	-	-
			90%	8,156	1,142	316	9	**5,710**	2.14	-	-	-
			75%	8,156	1,142	316	9	6,200	1.71	-	-	-
Modified Conventional	9-7	10%	95%	8,156	1,142	316	9	5,290	2.47	1.64	-	-
			90%	8,156	1,142	316	9	**5,680**	2.14	1.28	-	-
			75%	8,156	1,142	316	9	6,190	1.71	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	8,156	1,142	316	9	6,040	-	1.64	1.87	0.83
			90%	8,156	1,142	316	9	**6,210**	-	1.28	1.40	0.83
			75%	8,156	1,142	316	9	6,490	-	0.67	0.70	0.83

Table 13D. ACI 214.4R-10 Slab 1D Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	9,300	1,700	369	6	4,180	3.01	-	-	-
			90%	9,300	1,700	369	6	**5,070**	2.49	-	-	-
			75%	9,300	1,700	369	6	6,140	1.86	-	-	-
Modified Conventional	9-7	10%	95%	9,300	1,700	369	6	4,150	3.01	1.64	-	-
			90%	9,300	1,700	369	6	**5,040**	2.49	1.28	-	-
			75%	9,300	1,700	369	6	6,130	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	9,300	1,700	369	6	6,450	-	1.64	2.02	0.83
			90%	9,300	1,700	369	6	**6,780**	-	1.28	1.48	0.83
			75%	9,300	1,700	369	6	7,250	-	0.67	0.73	0.83

Table 13E. ACI 214.4R-10 Slab 1E Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	9,452	2,068	366	6	3,230	3.01	-	-	-
			90%	9,452	2,068	366	6	**4,300**	2.49	-	-	-
			75%	9,452	2,068	366	6	5,610	1.86	-	-	-
Modified Conventional	9-7	10%	95%	9,452	2,068	366	6	3,200	3.01	1.64	-	-
			90%	9,452	2,068	366	6	**4,280**	2.49	1.28	-	-
			75%	9,452	2,068	366	6	5,600	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	9,452	2,068	366	6	6,340	-	1.64	2.02	0.83
			90%	9,452	2,068	366	6	**6,740**	-	1.28	1.48	0.83
			75%	9,452	2,068	366	6	7,290	-	0.67	0.73	0.83

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Table 13F. ACI 214.4R-10 Slab 1F Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	9,550	1,695	369	6	4,450	3.01	-	-	-
			90%	9,550	1,695	369	6	**5,330**	2.49	-	-	-
			75%	9,550	1,695	369	6	6,400	1.86	-	-	-
Modified Conventional	9-7	10%	95%	9,550	1,695	369	6	4,410	3.01	1.64	-	-
			90%	9,550	1,695	369	6	**5,300**	2.49	1.28	-	-
			75%	9,550	1,695	369	6	6,390	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	9,550	1,695	369	6	6,660	-	1.64	2.02	0.83
			90%	9,550	1,695	369	6	**6,990**	-	1.28	1.48	0.83
			75%	9,550	1,695	369	6	7,460	-	0.67	0.73	0.83

Table 13G. ACI 214.4R-10 Slab 1G Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	8,663	1,123	393	6	5,280	3.01	-	-	-
			90%	8,663	1,123	393	6	**5,870**	2.49	-	-	-
			75%	8,663	1,123	393	6	6,570	1.86	-	-	-
Modified Conventional	9-7	10%	95%	8,663	1,123	393	6	5,220	3.01	1.64	-	-
			90%	8,663	1,123	393	6	**5,820**	2.49	1.28	-	-
			75%	8,663	1,123	393	6	6,560	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	8,663	1,123	393	6	6,250	-	1.64	2.02	0.83
			90%	8,663	1,123	393	6	**6,490**	-	1.28	1.48	0.83
			75%	8,663	1,123	393	6	6,840	-	0.67	0.73	0.83

Table 13H. ACI 214.4R-10 Slab 1H Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	8,200	1,925	321	6	2,410	3.01	-	-	-
			90%	8,200	1,925	321	6	**3,410**	2.49	-	-	-
			75%	8,200	1,925	321	6	4,620	1.86	-	-	-
Modified Conventional	9-7	10%	95%	8,200	1,925	321	6	2,380	3.01	1.64	-	-
			90%	8,200	1,925	321	6	**3,390**	2.49	1.28	-	-
			75%	8,200	1,925	321	6	4,610	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	8,200	1,925	321	6	5,420	-	1.64	2.02	0.83
			90%	8,200	1,925	321	6	**5,780**	-	1.28	1.48	0.83
			75%	8,200	1,925	321	6	6,300	-	0.67	0.73	0.83

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Table 13I. ACI 214.4R-10 Slab 2A Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	8,837	1,636	342	6	3,910	3.01	-	-	-
			90%	8,837	1,636	342	6	**4,760**	2.49	-	-	-
			75%	8,837	1,636	342	6	5,790	1.86	-	-	-
Modified Conventional	9-7	10%	95%	8,837	1,636	342	6	3,880	3.01	1.64	-	-
			90%	8,837	1,636	342	6	**4,740**	2.49	1.28	-	-
			75%	8,837	1,636	342	6	5,790	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	8,837	1,636	342	6	6,120	-	1.64	2.02	0.83
			90%	8,837	1,636	342	6	**6,440**	-	1.28	1.48	0.83
			75%	8,837	1,636	342	6	6,890	-	0.67	0.73	0.83

Table 13J. ACI 214.4R-10 Slab 2B Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	11,282	593	435	6	9,500	3.01	-	-	-
			90%	11,282	593	435	6	**9,810**	2.49	-	-	-
			75%	11,282	593	435	6	10,180	1.86	-	-	-
Modified Conventional	9-7	10%	95%	11,282	593	435	6	9,360	3.01	1.64	-	-
			90%	11,282	593	435	6	**9,700**	2.49	1.28	-	-
			75%	11,282	593	435	6	10,140	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	11,282	593	435	6	8,650	-	1.64	2.02	0.83
			90%	11,282	593	435	6	**8,810**	-	1.28	1.48	0.83
			75%	11,282	593	435	6	9,080	-	0.67	0.73	0.83

Table 13K. ACI 214.4R-10 Slab 2C Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	9,853	2,479	380	6	2,390	3.01	-	-	-
			90%	9,853	2,479	380	6	**3,680**	2.49	-	-	-
			75%	9,853	2,479	380	6	5,240	1.86	-	-	-
Modified Conventional	9-7	10%	95%	9,853	2,479	380	6	2,370	3.01	1.64	-	-
			90%	9,853	2,479	380	6	**3,660**	2.49	1.28	-	-
			75%	9,853	2,479	380	6	5,240	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	9,853	2,479	380	6	6,400	-	1.64	2.02	0.83
			90%	9,853	2,479	380	6	**6,870**	-	1.28	1.48	0.83
			75%	9,853	2,479	380	6	7,530	-	0.67	0.73	0.83

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Table 13L. ACI 214.4R-10 Slab 2D Compressive Strength Data

Method	ACI 214 Eqn.	Fractile	Confidence Level	f_c^{bar} (psi)	s_c (psi)	s_a (psi)	Sample Size (n)	Equiv. Strength (psi)	Factors (Tables 9.2 - 9.5)			
									K	Z	T	C
Conventional	9-6	10%	95%	10,635	1,229	410	6	6,940	3.01	-	-	-
			90%	10,635	1,229	410	6	**7,570**	2.49	-	-	-
			75%	10,635	1,229	410	6	8,350	1.86	-	-	-
Modified Conventional	9-7	10%	95%	10,635	1,229	410	6	6,870	3.01	1.64	-	-
			90%	10,635	1,229	410	6	**7,530**	2.49	1.28	-	-
			75%	10,635	1,229	410	6	8,330	1.86	0.67	-	-
Alternative Approach	9-8 and 9-9	10%	95%	10,635	1,229	410	6	7,820	-	1.64	2.02	0.83
			90%	10,635	1,229	410	6	**8,070**	-	1.28	1.48	0.83
			75%	10,635	1,229	410	6	8,450	-	0.67	0.73	0.83

3. Petrographic Examination

After additional (secondary) cores for compressive strength testing were deemed necessary, at least two cores were taken from each pour for additional petrographic examination. A total of 29 additional cores were selected for petrographic examination. Table 14 and Attachments 48-52 provide detailed information about the cores and Windsor probes petrographically reviewed by the APS (Attachment 47), TEC (Attachment 50A), UCT (Attachment 51), JTC (Attachment 52), and RJ Lee (Attachment 54) petrographic examinations.

The petrographic examinations found the overall quality of the concrete ranged from fair to good. The cores contained 3/8 to 1/2 inch coarse aggregate and GGBFS. The cement paste was hard and the paste-to-aggregate bond was characterized as moderately tight to tight. Both entrapped and entrained air was present in the cores. Air content near the top surfaces was generally lower than air content in the body of the core. The depth of carbonation was generally less than 1/16 inch. A 1.5 mm thick mortar layer was observed on one core.

Petrographic examination of the core samples determined that, in general, the concrete placed in the SSTC deck has entrapped air that varied between 0.7% and 6.1%. The presence of entrapped air in concrete can reduce its compressive strength.

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Table 14. Cores by Core Number and Pour Location Subjected to Petrographic Examination

Location	APS Initial	APS Supplemental	JTC	TEC	RJLG	UCT
Level 330						
1A	--	--	#58	#56	--	#57
1B	--	--	#70	--	--	#69
1C	#7	#10	#47	#39-42	--	#48
1D	--	--	#71	--	--	#75
1E	--	--	#96	--	--	#99
1Ea	--	--	--	--	--	--
1Eb	--	--	--	--	--	--
1F	--	--	#105	--	#11	#108
1G	--	--	#79	--	--	#86
1H	--	--	#93	--	--	#93
Pour Strip East	#17	--	--	--	--	--
Pour Strip West	--	--	--	--	#3	--
Level 350						
2A	--	#26, #28, #30	#112	--	#29	#116
2B	#31	--	#122	--	--	#123
2C	--	--	#127	--	--	#131
2D	--	--	#135	--	--	#141
Pour Strip East	--	--	--	--	--	--
Columns	--	--	#147	--	--	#143, #151

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4. Entrained Air Content Testing

Below find a summary of the entrained air content analysis. Air content allowed by most restrictive Contract Document requirement or approved mix is 5.5% ± 1.5%.

Table 15.

	APS ROM	APS Supplemental	JTC	TEC	UCT
Level 330					
1A			4%-6%		3.3%
1B			6%-8%		4.1%
1C			4%-6%		1.4%
1D			6%-9%		3.6%
1E			5%-7%		2.6%
1Ea	–	–	–	–	
1Eb	–	–	–	–	
1F			6%-8%		6.3%
1G			6%-9%		6.3%
1H			6%-9%		3.7%
Pour Strip East					
Pour Strip West					
Level 350					
2A	2.4% 3.2% 4.5%	3.3% 2.0% 2.5% 4.3%	4%-5%		1.6%
2B			5%-7%		5.2%
2C			6%-8%		5.9%
2D			6%-9%		6.6%
Pour Strip East				5.4% 6.8% 4.2% 8.2%	
Column B10			2%-3%		
Column B2					2%
Column C7					1.7%

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5. Entrapped Air Volume

Below find a summary of the entrapped air volume (no limitation in the Contract Documents).

Table 16.

	APS Initial	APS Supplemental	JTC	TEC	UCT
Level 330				–	
1A	–	–	–		1.3%
1B	–	–	–		1.6%
1C	–	–	–		6.1%
1D	–	–	–		4.9%
1E	–	–	–		3%
1Ea	–	–	–		–
1Eb	–	–	–		–
1F	–	–	–		0.3%
1G	–	–	–		3.9%
1H	–	–	–		
Pour Strip East	–	–	–		
Pour Strip West	–	–	–		
Level 350				–	
2A	0.1%-2.6%	–	–		2%
2B	–	–	–		0.7%
2C	–	–	–		1.0%
2D	–	–	–		0.6%
Pour Strip East	–	–	–		
Col. B2	–	–	–		2.8%
Col. C7	–	–	–		0.3%

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6. Air Void Spacing (not a Contract Document requirement)

Below find a summary of the air void spacing analysis vs. ASTM C856, ACI 201.2R-92, and ACI 212.3R-91 recommendations.

Table 17.

	APS Initial	APS Supplemental	JTC	TEC	UCT
Level 330	0.017-0.008	–	–	0.021-0.011	
1A		–	–		0.016
1B		–	–		0.007
1C		–	–		0.028
1D		–	–		0.014
1E		–	–		0.018
1Ea		–	–		0.011
1Eb		–	–		
1F		–	–		
1G		–	–		0.010
1H		–	–		0.015
Pour Strip East		–	–		–
Pour Strip West		–	–		
Level 350		–	–		
2A		–	–		0.022
2B		–	–		0.010
2C		–	–		0.008
2D		–	–		0.010
Pour Strip East		–	–		
Column B10		–	–		
Column B2		–	–		0.032
Column C7		–	–		0.025

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7. Water/Cement Ratio

Below find Table 18, a summary of the water/cement ratio analysis of the totality of the cores taken.

Maximum water/cement ratio allowed by the most restrictive Contract Document or approved mix is 0.29 (vs. Contract Document 0.40).

Table 18.

	APS ROM W/C	Unhydrated Cementitious Material	APS Supplemental	JTC	Unhydrat ed	TEC	UCT	Unhydrated Cementitious Material
Level 330						0.35-0.41		
1A	0.39-0.44	10%-12% un-hydrated cement 16% - 18% un-hydrated slag	—	0.40-0.45			0.40 ± 0.05	6%-11%
1B			—	0.38-0.43			0.40 ± 0.05	6%-11%
1C	0.39-0.44	10%-12% un-hydrated cement 16% - 18% un-hydrated slag	—	0.38-0.43			0.40 ± 0.05	5%-12%
1D	—	—	—	0.35-0.40			0.40 ± 0.05	8%-13%
1E	—	—	—	0.35-0.40			0.40 ± 0.05	7%-13%
1Ea	—	—	—	—	—	—	—	—
1Eb	—	—	—	—	—	—	—	—
1F	—	—	—	0.35-0.40	—		0.40 ± 0.05	8%-13%
1G	—	—	—	<0.38	—		0.40 ± 0.05	8%-12%
1H	—	—	—	0.35-0.40	—		0.40 ± 0.05	7%-12%
Pour Strip East	—	—	—		—		—	
Pour Strip West	—	—	—		—		—	
Level 350								
2A	0.38-0.43 0.39-0.49 0.38-.043	10%-12% un-hydrated cement 16% - 18% un-hydrated slag	—	0.35-0.45	—		0.40 ± 0.05	8%-13%
2B	—	—	—	0.35-0.40	—		0.40 ± 0.05	7%-11%
2C	—	—	—	<0.38	—		0.40 ± 0.05	7%-12%
2D	—	—	—	0.35-0.40	—		0.40 ± 0.05	8%-13%
Pour Strip East	—	—	—		—			
Column B10	—	—	—	<0.38	—			
Column B2	—	—	—		—		0.40 ± 0.05	6%-12%
Column C7	—	—	—		—		0.40 ± 0.05	8%-13%

B. Grout Testing Results Summary

APS also performed petrographic examination, pH analysis, sulfate analysis, and chloride content analysis on 27 grout samples. The grout samples were taken from tendon ducts at beams (7), girders (4), and slabs (16).

APS's grout materials testing report is included in Attachment 53, which summarizes the grout sample locations and tests that were performed at each location**.**

An overview of the grout materials requirements and testing results is as follows.

1. Grout design information

Mixture of Portland cement, water, and admixtures.

a. Compressive strength at seven days - 8,000 psi minimum
b. Shrinkage - maximum of 0.5 percent
c. W/c ratio - maximum of 0.40

2. Petrographic examination

Overall quality of the grout was characterized by APS as "good". Paste hardness was characterized as "medium to hard". Carbonation depth at Sample S13 was up to 1/32 inch. Trace amounts of gypsum were observed in Samples B01, B05, and G03. The presence of gypsum in trace amounts was confirmed by APS using X-ray diffraction but its presence not an issue.

3. Grout test results are presented below.

a. pH analysis - values ranged from 12.72 to 12.91.
b. Sulfate content analysis - values ranged from 2.09 to 2.68 percent by mass of sample.
c. Chloride content analysis - values ranged from negligible to 0.029 percent by mass of sample.

(There are no Contract Document requirements for grout material.)

C. Durability Tests

1. Porosity Testing

Porosity is a measure of the volume occupied by voids in a material and is measured as a volume percentage. The porosity was determined in accordance with ASTM C642. The average porosity is the average of the three cores was 13%. This is a measure of permeable voids in the concrete samples excised from the deck, and is used as an input into the Service Life Model (Stadium©). There are no specifications or requirements for porosity in the Contract Documents to compare these values against.

2. Migration Testing

This test method is used to evaluate the diffusion coefficient of ionic species in cementitious materials. This test method was performed using a modified version of the AASHTO T259 and ASTM C1202-97 standard test procedures. The input parameters for the service life model require that a modified test version of ASTM C1202 be performed. The service life model was calibrated based on values obtained from this modified test method.

The test method consists of monitoring the intensity of electrical current passing through a cylindrical test specimen during a 10- to 15-day testing period. The numerical results are the current intensities recorded during testing. These results provide information required to evaluate the ionic diffusion coefficients, which is a controlling material property for performing a numerical simulation of chloride ion diffusion through a porous medium such as concrete.

3. Drying Testing

This test method is used to determine the moisture transport coefficient of cementitious materials by measuring the mass loss due to evaporation and moisture transport in specimens exposed to constant temperature and relative humidity.

The following moisture transport parameters used in the service model were determined by the results of the drying test:

a. Permeability (k)
b. Relative Permeability Factor (n)
c. Isotherm parameter B
d. Isotherm parameter C

The isotherm parameters B and C are estimated on the basis of the equilibrium value of moisture content in the 10 mm [0.4 in] test specimens. These parameters account for the variability of diffusion coefficients as temperature varies within concrete. The mass loss analysis provides the permeability and relative permeability parameters.

The drying tests also showed variability within the two samples for each of the Closure Pours and cores. The drying test laboratory simulation combines the results of the drying test and the migration laboratory simulation.

D. <u>Engineering Analysis of Concrete Materials Testing Results</u>

1. The overall quality of the concrete was characterized as ranging from “good” to “fair to poor.” The noted GGBFS and coarse aggregate size are consistent with the approved mix design for the non-self-consolidating concrete mixes. The amount of cementitious material in the mix is relatively high. This, combined with relatively small aggregate size, could result in significant early age shrinkage (i.e. autogenous and drying shrinkage). For clarity, autogenous shrinkage occurs in some concrete mixes with low water/cement ratios (less than 0.42). Water is drawn into the hydration process and the demand for

more water creates very fine capillaries. Surface tension within the capillaries can lead to cracking. The estimated water/cement ratio varied around the maximum specified (0.40) in the Contract Documents and greater than the approved mix design of 0.29.

2. The petrographic examinations of the concrete samples extracted from the deck yielded variable water/cement results depending on the location sampled. It is our opinion that the variability of the water/cement ratio is related to the addition of water to individual truck batches of concrete for workability during construction as described herein.

3. Air Content and Void Spacing

a. Air contents measured in cores taken from the deck was lower than the range that was specified in the Contract Documents.

Although not a Contract Document requirement, air void spacing greater than 0.008 inches, which, per ASTM standards is considered to be an upper bound value for “durable” concrete. The air void spacing varied from 0.008 to 0.032. See Tables 16 and 17.

These results appear to be consistent with previous CTL Group, Inc. (CTL) petrographic examination results (they performed no compressive strength testing, only testing to determine durability issues due to the concerns expressed by WMATA regarding placement of the specified evaporation retarder). CTL did note issues with the durability of the slab concrete due to lack of sufficient entrained air in the top 30 millimeters of the slab.

b. Entrapped Air – Table 16 reflects the amount of entrapped air (no limit in Contract Documents).

4. Water/Cement Ratio and Unhydrated Cementitious Material

The water/cement ratio is substantially higher than the approved design mix and is indicative of water in the mix as delivered being added at the site see Table 18.

5. Core Strengths versus Cylinder Strengths

Concrete cores excised from the SSTC slabs exhibited significantly lower compressive strengths when compared to compressive strengths measured via cylinder as reported.

As noted earlier, there are a number of different factors that can cause a difference between concrete that is placed in a structure and the concrete samples that are made at the time of construction and intended to be representative of the in-situ concrete.

Compressive strength of concrete and the water/cement ratio of concrete are codependent. When water is added to the concrete (increasing the water/cement ratio), the compressive strength is reduced. Since samples were only taken on one in five trucks during construction (consistent with Industry Standard), cylinders were not made for the majority of the concrete placed on the deck. As a result, concrete core

test results from the slabs do not necessarily correspond with test results of cylinders that were made for a specific concrete pour.

Concrete must be placed in an environment that is warm enough for the chemical reaction that gives concrete strength (known as hydration) to occur.

A number of pours began and continued when temperatures were substantially less than 32°F.

Petrographic examinations of the concrete cores from the slabs indicate that unacceptable percentages of the Portland cement and slag were unhydrated. This observation is consistent with concrete experiencing a temperature low enough to slow hydration to the point that the available water dried out before the cement could hydrate.

The concrete cylinders after being molded, we are told, were stored in curing boxes, and therefore experienced a substantially different environment than the concrete in the slabs.

Compressive strength testing of concrete core specimens removed from the structure has yielded a wide range of results. The composition of any concrete varies (non-homogeneous), and a concrete's compressive strength can be affected by numerous factors that start with variability in manufacture and procurement of cementitious materials and aggregate, mixing and transport, addition of water on site in varying amounts from truck to truck, placement methods, finishing, ambient conditions at the time of placement, and curing.

The apparent randomness of the strength values yielded by testing of the cores taken from the completed structure is suggestive of water being added and insufficient and inconsistent quality control. Specific additional contributing factors may have included:

- Areas where concrete was left exposed longer before curing blankets were placed
- Excessive movement (finishing) of concrete during placement
- Delayed finishing
- Inconsistent heating during curing
- Arrested hydration due to cold weather conditions.

6. Chloride

Low initial (cast-in) chloride content makes early service life corrosion of steel unlikely, provided that the encasing concrete is free of cracks. Once the structure is placed into service, the concrete will be exposed to environmental chlorides, primarily from seasonal application of de-icing salts or those transported onto the deck by vehicles.

XI. Service Life Modeling

Our analytical work included use of the computer software suite STADIUM©, a software application produced by Simco Corporation. STADIUM© simulates the transport of ions in cementitious materials. The model consists of two modules: the "transport" module, which controls the kinetic of exchange (ion movement) and the chemistry module, which considers the reaction of ions with the cementitious binder. Chlorides react with concrete as they penetrate the concrete, which affects rate of penetration into the concrete.

The STADIUM© model takes a number of factors into account that affect how chlorides propagate through the concrete cover to the reinforcement. These factors are calibrated based on the results of the laboratory materials testing of samples excised from the structure as taken by RJ Lee. Detailed inputs into the service life model are summarized in Attachments 54 and 54A) and include the concrete mix design, cement chemistry, transport properties determined by laboratory testing, and hydration of the cementitious material. Note that numerical simulations are performed with the assumption that the concrete mixtures are initially unaltered (i.e. free of defects or any cracks), i.e., the fact that the service life model does not calculate times to corrosion initiation that account for the presence of cracks or other defects in the concrete. Estimates of service life are only valid at locations where concrete is uncracked. As discussed in the report, at locations where the concrete is cracked, chlorides are able to access unprotected reinforcement immediately, and therefore corrosion is considered to initiate immediately at these locations.

A. Ion Exposure

Exposure to deicing salts is modeled as a temporary and periodic ionic exposure, defined as a sinusoidal function centered on the coolest day of the temperature cycle. The ionic exposure duration used in the model is shorter than the actual period of exposure in order to take into consideration days without exposure to deicing chemicals, vehicle circulation, and variations in the exposure concentrations. The relative humidity associated with this exposure duration is by default set to 100%.

B. Temperature and Humidity

Values for temperature and humidity were obtained from the database contained in STADIUM©. The values in this database were obtained from publicly available weather data for the site geography (specific to the Baltimore, MD area). The exposure data is estimated based on published databases included in the software package that consider typical exposure conditions in exposed structures and available information on deicing applications. The exposure data were estimated based on typical wetting and drying experienced in exposed structures.

C. Corrosion Thresholds

A corrosion threshold corresponds to a critical ionic concentration required to initiate corrosion of the reinforcing bars. STADIUM© includes two types of corrosion thresholds: the chloride concentration and the Cl^-/OH^- (chloride to alkalinity) ratio thresholds. The values of these thresholds have been extensively investigated during the past decade.

Corrosion of reinforcing steel embedded in concrete is an electrochemical process that causes the concrete to act like a battery. The steel becomes the anode in the electrical circuit, and the concrete acts as an electrolyte. Freshly placed concrete with low chloride ion content is an intentionally poor electrolyte, providing a healthy, electrochemically passive environment that protects the reinforcing steel. As the concrete ages, chloride ingress and other chemical changes in the concrete as it continues curing that lower pH improve the concrete's electrolytic properties, increasing the risk of steel corrosion. The specific thresholds at which corrosion is likely to initiate, and the time frame in which these thresholds are likely to be reached under different service conditions, have been extensively researched, and the findings of this research have been incorporated into proprietary analytical software packages such as STADIUM©.

For epoxy-coated steel, the chloride content for corrosion initiation is considered the same as for black steel (i.e. 500 ppm). Since the coating is damaged (aka holidays or nicks), corrosion initiates at the same chloride level and can cause localized pitting in the exposed area, which can ultimately be even worse than widespread corrosion. Because the concrete mix design includes a corrosion inhibiting admixture, the threshold for initiation of corrosion is increased as the concrete can resist a higher amount of chloride ions before corrosion initiates. In concrete without corrosion inhibitor the threshold for initiating corrosion is taken as 500 ppm. For concrete containing corrosion inhibitor, per the approved mix design used in the deck pours, the corrosion initiation is considered to be 1,500 ppm.

D. <u>Service Life Model Results</u>

Three simulations were performed to model the variability in the diffusion coefficients obtained from the laboratory testing. Due to the variability in the concrete test results, a worst case and a best case scenario were used for the laboratory modeling. These scenarios were accounted for in the parameters input into the STADIUM© durability model. The terms High and Low refer to the highest and lowest values, respectively, for the migration coefficients obtained from the samples tested for the Project. They do not refer to predefined "High" and "Low" values. The values obtained for High, Average, and Low were 12.04, 7.37, and 2.32, respectively. The time for corrosion initiation (in years) at different depths is summarized below.

Table 19. Summary of Service Life Model Results

MODEL	**CONCRETE COVER**			
Depth / **# of years to corrosion onset**	**0.5 IN.**	**1.0 IN.**	**1.5 IN.**	**2.0 IN.**
High Diffusion Coefficient	8	18	35	> 50
Average Diffusion Coefficient	10	32	> 50	> 50
Low Diffusion Coefficient	34	> 50	> 50	> 50

The Service Life model calculates the time it takes for chloride ions to propagate through the concrete to the depth of the reinforcement. The diffusion coefficient describes the rate at which chloride ions are able to travel through a porous material (such as concrete).

From the table, it can be seen that even for the higher diffusion coefficient values, reinforcing steel placed with a concrete cover of 2 inches or greater will not start corroding during the first 50 years in un-cracked concrete.

We point out that ADOJAM's previous study is fundamentally incomplete in that it does not represent the in situ material properties, but rather "representative similar" concretes used in other construction in the region. Furthermore, they neglect to report time to corrosion for covers less than 1 inch, which were noted at exposed post-tensioning locations, column reinforcing, etc.

Where post-tensioning ducts are exposed, the sheathing can be compromised by vehicular traffic and/or salts. Once the sheathing is compromised, the time to corrosion initiation would be the same as mild reinforcement with the same cover. In this scenario, the cover would be that provided by the grout in the duct. In this case, corrosion would initiate immediately once the sheathing and grout are compromised. In addition, due to the prestress force in the cables, they are subjected to additional stress corrosion, which is more aggressive (increased corrosion rate) than chloride corrosion.

XII. METHODOLOGY OF AS-BUILT REVIEW

We utilized the documents forwarded to us, our field observations, our analyses, and our laboratory materials testing to assess strength and serviceability of the as-built structure.

A. Material Properties and Assumptions

The materials properties that were used in the as-designed analysis are as follows:

1. Cast-in-place Concrete Material

a. ACI 318-02 provides the regulations for acceptance of concrete (i.e., of sets of three cores, none can be lower than 75% f'c and average has to be greater than 85%). Using that requirement, there are pours that do not have acceptable concrete per the Contract Documents (Tables 10, 10A, and 11).

b. For the as-built analyses, we initially used two concrete compressive strengths: 8,000 as-designed and 5,000 psi from the ROM APS results.

c. After our secondary core test results, we also analyzed the core compressive strength test results using ACI 214.4R-10 statistical analysis. Since there is no universally accepted method for determining the 10 percent fractile of in-place concrete strength, we evaluated three separate calculation methods (Tables 12A, 12B, and 12C):

- Section 9-6: conventional
- Section 9-7: modified conventional
- Sections 9-8 and 9-9: alternative).

The compressive strength calculation results varied. Based on this information, we used a compressive strength equal to 6,970 psi for our reanalysis as shown below.

(For initial stress calculations, we assumed that the compressive strength at the time of stressing of the post-tensioning was 75 percent of the design strength, of 8,000 or 6,000 psi.)

2. Conventional mild reinforcing steel bars

Based on our review of the mill test results, the reinforcing bars comply with or exceed the American Society for Testing and Materials (ASTM) A615 or A706 requirements as required by the Contract Documents. Therefore, we used 60,000 psi as the yield strength of mild reinforcing bars.

3. Pre-stressing strands

The terms “prestressing wire” and “post-tensioning wire” (aka strands) are used interchangeably in the industry vis-à-vis strength requirements for various specified material and other characteristics. E.g., Prestressing wires used in precast concrete are installed before the pour and stressed and prestressing wires in post-tensioned concrete are put in sheathing before the pour and stressed after the concrete is poured.

Based on our review of the mill order test results, the prestressing strand material conforms or exceeds the requirements of ASTM A416 as required by the Contract Documents. Based on the information above, we used 270,000 psi as the tensile strength of all prestressing strands (wire).

4. Slab Thickness

We would note that based on the Rice survey, the slabs on Level 330 and Level 350 are less than Contract Document-allowed minimum thickness of 9-3/4 inches.

In our calculated load-carrying analytical work, we modified the thickness of the elevated slabs to conservatively envelop the slabs based on the Rice survey thickness. We modeled “thin” slab regions as 8-1/2-8-3/4 inches thick for 8,000 psi concrete and 9 inches thick for 6,970 psi concrete, which, based on our engineering judgment and based on the ACI 214.4R-10 analysis method, was selected as a nominal conservative value for “thin” slab areas. Areas of thin slab thickness are isolated and are not representative of large areas of the slab. “Thick” slab regions were modeled as 10 inches thick. Based on our engineering judgment, this value was selected as a nominal conservative value for “thick” slab areas.

We evaluated slab thickness added dead load beyond 10-3/8” up to 12-1/4” but we found no impact to beams, girders, or slabs in shear, torsion, or moment for concrete or for the as-designed and as-built concrete.

5. Tendons

Tendon forces were calculated based on our review of the post-tensioning stressing records. First, we reviewed the measured strand elongations. Second, we compared the field measured elongations to VSL’s calculated elongations. Third, we calculated the average initial stress, service level, and average final force in the tendons. We used the calculated tendon forces in our analytical models. In general, the tendon forces in our calculated model were similar to or slightly less than the jacking forces shown on the VSL shop drawings. Attachment 55 summarizes the number of as-built tendon elongations in each pour that did not comply with VSL’s calculated target elongation range of ±5%, modified to 7% by PB, a PTI-recommended standard.

Where tendon profile information was measured, those profiles were used in our as-built analysis.

B. <u>As-Built Analysis Methods</u>

Analysis methods included hand calculations, SAP 2000© V14.2.4, ADAPT-PT© 2010, ADAPT-EDGE© 2012, and CSI Column© V8.3.2 computer simulations. Hand calculations were typically used to assess members that were not post-tensioned or to verify our modeling assumptions. All analyses were performed based on the IBC 2003, ACI 318-02, and the WMATA Manual of Design Criteria as referenced in the Contract Documents.

The following sections describe the SAP©, ADAPT©, and ADAPT-EDGE© structural analysis models.

1. Limit States

As required by ACI 318-02, three loading conditions (limit states) were to be analyzed:

- <u>Initial</u> - includes unfactored concrete self-weight and post-tensioning forces.
- <u>Service</u> – includes all unfactored loads (concrete self-weight, superimposed dead load, live load, snow load, and lateral load) and post-tensioning forces.
- <u>Ultimate</u> - includes all factored design loads (concrete self-weight, superimposed dead load, live load, snow load, and lateral load) and hyperstatic forces. Note that hyperstatic forces are forces resulting from support restraint of a post-tensioned element.

a. SAP 2000© Models

SAP 2000© V14.2.4 is a general-purpose finite element analysis program developed by Computers and Structures, Inc. (CSI). Three SAP 2000 models were created - one of Level 330, one of Level 350, and a combined model for the entire building (Attachment 56). These models were used to calculate the beam and girder internal force envelopes (i.e. maximum shears and moments) based on the application of total dead, live (moving trucks and people), and snow loads. The resulting force envelopes were used to verify that we used in ADAPT-PT© and hand calculations utilized the worst case loading conditions for the element under consideration.

In the model, truck loads were applied to the structure on one or more predefined paths that simulate the specified slab marking/lane plan as shown on Sheets A2.11 and 2.12 of the Contract Documents.

Finally, the combined model was used to assess column reinforcing steel requirements.

b. ADAPT-PT© Models

ADAPT PT© 2010 is an Industry Standard, two-dimensional, post-tensioning analysis and design program owned by ADAPT© corporation. The program allows the user to define element material properties, geometry, and loading conditions. Results include, but are not limited to, concrete stresses under initial and service loads, post-tensioning force requirements, and mild steel reinforcing requirements for the ultimate strength limit state. ADAPT-PT© was used to analyze selected post-tensioned slabs, beams, and girders.

c. ADAPT-EDGE© Model (Finite Element)

ADAPT-EDGE© 2012 is a three-dimensional, finite element, post-tensioning analysis and design program also owned by ADAPT© corporation. The program allows the user to analyze multi-story buildings for gravity and post-tensioning loads, and includes some limited capabilities regarding lateral load analysis. Results include, but are not limited to, resultant stresses under initial and service load states, deflections, and resultant axial, shear, and moments for the ultimate strength limit state.

We used ADAPT-EDGE© to calculate internal concrete stresses of post-tensioned slabs, beams, and girders using the as-designed and as-built information available (slab thicknesses, concrete strengths, 8,000 (per the Contract Documents), then based on the ROM strength of 5,000 psi, and finally with secondary core strength results in representative areas with 6,970 psi, tendon elongations, tendon profiles) for areas of the structure with complex geometry.

Because ADAPT-EDGE© does not contain algorithms to generate moving load cases automatically, truck loads were placed based on the results of the SAP

2000© influence line analyses, done with SAP 2000© and the resultant output was compared with the ADAPT-PT© results.

d. CSI Column©

CSICOL© V8.3.2, a software package owned by CSI, was used for the analysis of selected concrete columns. Where necessary, the geometry of the cross-section was customized to account for unique conditions including the lack of concrete cover or relocated reinforcing bars.

C. As-Built Analysis Results

1. Slab Pour Strips

Hand calculations were performed for the slabs at the closure pours on Level 330 assuming 8,000 psi, 5,000 psi (ROM), and 6,970 psi concrete. Results of an analytical 4.8-foot wide strip indicate that the slabs at these locations, as built, do not have sufficient shear or flexural capacity to support the design loads. Note that this 4.8-foot strip width corresponds to PB's original strip width and AASHTO requirements for slabs with reinforcement parallel to traffic but is conservative since the reinforcement is perpendicular to traffic. When the concrete strength is reduced to 6,970 psi as above, the insufficiency remains.

2. Post-Tensioned Slabs

The elevated slabs were evaluated using ADAPT-PT© under the following parameters:

- As-built tendon geometry and force. For each bay examined, the average vertical drape of the as-built tendons in that bay was used in the analytical model.
- Concrete compressive strengths of 8,000 psi (specified compressive strength) and 5,000 psi (based on ROM core compressive strength) were initially assumed as upper and lower bounds in the as-built analysis. The low core strength results were later reviewed to determine the in situ strength based on ACI 214.4R-10 analysis using the secondary core results, which yielded 6,970 psi.
- Slab thickness of 10 inches and 8-1/2 inches (lower bound "thin" slab area) were initially assumed as upper and lower bounds for the as-built analysis. Note that although the slab surveys did observe thicknesses that were less than 8-1/2 inches, these areas were small relative to the total slab area, and would not be indicative of overall slab load-carrying capability.
- Multiple truck load patterns were intended to induce maximum positive moment, negative moment, and shear in the one-way design strips.
- Assuming design strip width of 4.8 feet, corresponding to PB's original design strip width and AASHTO requirements for slabs with reinforcement parallel to traffic. Due to the limited strip width, truck wheel loads rather than axle loads were applied to the design strip. The design truck wheels are spaced at six feet on center, making only a single line of wheels tributary to a 4.8-foot wide strip.

Results for these models under the 8,000 psi Contract Document requirement, 5,000 psi ROM, and 6,970 psi concrete indicate the following:

- As-built as-designed balanced force in the slabs exceeds 100 percent of the dead load.
- As-built tensile stresses under initial and service loading conditions exceed 3√f'c and 6√f'c, respectively, on both elevated levels of the structure. Note that ADAPT-PT© analyzes one-way slab behavior, which may not accurately predict slab behavior at low, service (unfactored) load levels when the slab is uncracked. Therefore, ADAPT-EDGE© was also used to assess service level stresses, as described later in this section.
- The 4.8-foot wide, 10-inch thick slab strips have sufficient capacity to resist the design moments and shears.
- The 4.8-foot wide, 8-3/4 inches thickness analysis for 8,000 psi or 9 inches thickness for 6,970 psi strips have sufficient moment capacity but do not have sufficient shear capacity to resist the superimposed design loads.

Reducing the concrete strength from 8,000 to 6,970 psi only exacerbates each of the conditions described above. Our analysis indicates that the minimum acceptable slab thickness for shear flexure and torsion strength is approximately 9 inches ± 1/4", using a concrete strength of 6,970 psi and a design strip width of 4.8 feet.

After completing the ADAPT-PT© analyses, we used the available as-built information (slab thickness, tendon profiles, tendon forces, etc.) and ADAPT-EDGE© to perform in a more general assessment of the post-tensioned slabs. For clarity, slab analysis with this model is valid only until the slab cracks. After the slab cracks, the model is not capable of redistributing the loads properly. Since this program does not allow for input of moving loads, we placed trucks in various load patterns as determined from the SAP 2000© model.

Results from this analysis indicate the following:

- Initial tensile stresses in the slabs are by design greater than 3√f'c. The largest initial tensile stresses were located on the bottom of the slab at supports and on the top of the slab at mid-span.

The predicted tensile stresses in ADAPT-EDGE© are significantly lower than the ADAPT-PT© values due to the slab elements used in the ADAPT-EDGE© model. Specifically, the ADAPT-EDGE© slab elements allow for two-way bending, assume no cracking, and distribute the applied loads over a larger area than the 4.8-foot wide one-way strip we used in the ADAPT-PT© model. This condition only exists in an analytical failure mode, which is inconsistent with the PB design parameters, e.g., one-way slab, as stated on the Contract Documents.

3. Beams

a. Pour Strip Beams

Using loads from the slab Pour Strip analysis described above, hand calculations were performed of the conventionally reinforced and partially post-tensioned beams below the Pour Strips. (These beams are post-tensioned at each end but not at mid-span.) Results indicate that these beams have sufficient shear and moment capacity to resist the design loads under 6,970 psi.

b. Post-Tensioned Beams

i. ADAPT-PT© Analysis

Six randomly selected representative multi-span continuous beam runs were selected for ADAPT-PT© analysis. Three of the beams were located on each elevated level. Beam runs subject to field investigation were selected for the as-built analysis based on the magnitude of the observed tendon deviation and location of the beam relative to the truck loads. In locations where as-built tendon drape information was not available due to the limits of the equipment, the VSL defined drapes were used.

The beams were modeled as "T-beams" as noted on the Contract Documents. Beam ends integral with the Pour Strips were modeled as restrained against rotation, while beam ends on the East (North) and West (South) sides of the expansion joints were modeled as unrestrained against rotation, since a slide bearing was detailed on the Contract Documents in that location. Intermediate beams were analyzed as partially restrained based on relative stiffness of the adjacent spans.

The beams were originally analyzed first for 8,000 psi and then for 5,000 psi based on the ROM compressive test results. As stated previously, selected areas were later re-examined to determine the effect of increasing the initial lower-bound compressive strength estimate from the ROM 5,000 psi to 6,970 psi as determined by ACI 214.4R-10.

Multiple load arrangements for the beams were evaluated by positioning trucks in selected spans to maximize shear, positive moment, negative moment, and torsion. Skip loading patterns were considered. In spans loaded by trucks, one truck was placed on either side of the beam simultaneously to maximize the truck load contributing to the individual beam load.

Results indicate that many of the beams have initial and service stresses that are higher than $3\sqrt{f'c}$ and $6\sqrt{f'c}$, respectively. These overstresses occur primarily where the continuous beams cross the supporting

girders, but also occur to a lesser degree at mid-span of the beams. The predicted overstresses are sufficiently high to result in cracking of the beams.

At ultimate load conditions, the beams were found to have sufficient flexural strength. However, in general, beams with spans longer than 28 feet center-to-center span in the travel lanes were calculated to have insufficient shear reinforcement for the ultimate demand with 8,000 psi concrete. The shear capacity at 6,970 psi would, by inspection, also be insufficient.

Finally, we investigated the impact of tendon deviations near the low point of the tendons. Results indicate that the beams analyzed had sufficient strength to resist the applied design loads, except long beams greater than 28-foot span in drive aisles. In addition, the beams appeared to be relatively tolerant of assumed low point deviation.

ii. Flexure and Shear Calculations

Eight multi-span continuous beam runs were analyzed using SAP 2000©, ADAPT-PT©, and Microsoft© Excel spreadsheets. Member forces were determined using SAP 2000© while ADAPT- PT© was used to estimate the hyperstatic forces. Finally, Microsoft© Excel spreadsheets were used to assess shear at each end of each beam as well as moment at each end and the middle of each beam. Results indicate that the beams have adequate flexural capacity but the longer drive aisle beams are understrength in shear with 8,000, 6,970, or 5,000 psi concrete.

iii. Combined Shear and Torsion

Our beam analyses described above did not initially include torsional loads. Typical shear, moment, and torsion have now been analyzed on several interior and perimeter beams were calculated using SAP 2000© while hyperstatic forces were estimated using ADAPT-PT©.

In order to calculate the ultimate torsion demands in the beams, slab area elements were modeled with reduced in-plane and out-of-plane stiffness. The stiffness reductions were used to approximate structural behavior at ultimate (factored load) demand levels and generally increased the torsion demands on the beams. In short, the slabs would not assist the interior beams in resisting torsion.

The perimeter (spandrel) beams are typically reinforced with closed stirrups, but the provided stirrups do not provide sufficient shear and torsion capacity to resist the applied loads. The interior beams typically have open U-shaped stirrups. These stirrups do not meet ACI 318-02 prescriptive requirements for torsional resistance and the applied torsional moment is greater than the threshold torsional moment.

Therefore, the interior beams do not comply with ACI 318-02 requirements for torsion under a concrete strength of 8,000 or 6,970 psi.

4. Girders

a. ADAPT-PT© Analyses

Girders on Level 330 (PG 5 and PG 9 through 12) and girders on Level 350 (PG 43 through 44 and PG38B) were selected based on their loading and geometry. The selected girders include both single span and continuous three-span configurations.

Results indicate that many of the girders have initial and service stresses that are higher than 3√f'c and 6√f'c, respectively. Results also indicate that the girders have insufficient capacity to resist the design torsion loads (using 8,000 psi concrete strength, and therefore by extrapolation, also 6,970 psi concrete).

Finally, we also investigated the impact of tendon deviations near the low point. Results indicate that the girders are relatively tolerant of the measured deviations as great as 5 inches (larger deviations were not generally considered in the sensitivity analysis since they were not observed during our field investigation).

b. Flexure and Shear Calculations

We analyzed the girders using forces from the SAP 2000© and hyperstatic forces from ADAPT-PT©. Analysis results indicate that the girders (using 8,000 and 6,970 psi concrete strength) had sufficient flexural and shear capacity to resist the design loads.

c. Combined Shear and Torsion

The girder calculations described above did not initially include torsional loads. Design loads, including torsion, were developed using SAP 2000© while hyperstatic forces were estimated using ADAPT-PT©. Our hand calculations and MathCad© spreadsheets indicate that the girder torsion loads exceed the threshold torsion allowed. Therefore, closed stirrups are required.

In order to calculate the ultimate torsion demands in the girders, slab area elements were modeled with reduced in-plane and out-of-plane stiffness. The stiffness reductions were used to approximate structural behavior at ultimate (factored load) demand levels and generally increased the torsion demands on the girders. In short, we estimated the maximum loads on the girders that caused torsion, neglecting slab stiffness.

In general, the girders have either Type 1 or Type 2 stirrups near mid-span. Type 1 stirrups are open U-shaped stirrups with two legs while Type 2 stirrups

are closed stirrups with two legs. The girders with Type 1 stirrups do not comply with ACI 318-02 prescriptive requirements for closed stirrups. In addition, the Type 2 stirrups do not provide sufficient shear and torsion strength to resist the applied loads. Finally, calculations performed at the ends of several girders indicate that the stirrups provided are not sufficient to resist the applied shear and torsion loads.

5. Columns

Based on our as-designed analysis and core compressive strength results, and nondestructive testing results, the as-built columns have sufficient capacity to resist the applied design loads.

XIII. Fire Rating

Architectural drawings A0.04 (Attachment 57) note that the building permit issued for this Project classifies this building as Type IA, which requires two-hour or three-hour fire ratings for various structural elements.

However, a reference on the same drawing notes a one-hour fire rating as being "supplied" for all structural elements, which is not consistent with the Code requirements.

The Code variances issued for the building as shown on sheet A-0.05 (Attachment 58) do not change the building classification or fire rating requirements, nor do the meeting minutes also noted on Architectural drawings. The permit was issued based on building construction Type IA, and therefore the fire rating required for various elements is two or three hours based on IBC 2003.

IBC Table 601 (Attachment 59) for a Type IA building type requires elements to have a two-hour or three-hour minimum fire rating for various structural elements as noted.

To achieve this, IBC prescribes that structural elements must be constructed with a minimum thickness for elements and an average minimum concrete cover for the positive moment reinforcement. (Positive moment reinforcement is on the bottom side of beams and slabs.) Absent actual Underwriters' Laboratory testing to evaluate the existing assembly's fire rating, these minimum cover requirements for the SSTC structural elements must be met as summarized in Tables 20A and 20B below.

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To achieve a three-hour fire rating for various elements, the minimum requirements are:

Table20A. Summary of Minimum Thickness Requirements for Three-Hour Fire Rating Construction[1]

Structural Element	Minimum Requirements		IBC 2003 Reference
Slabs	Thickness	5 inches	Tables 721.2.2, 721.2.3, and 721.2.4
	Cover	3/4 inches	Tables 721.2.2, 721.2.3, and 721.2.4
Beams and Girders	Thickness	8 inches	Tables 721.2.2, 721.2.3, and 721.2.4
	Cover	1-1/2 inches	Tables 721.2.2, 721.2.3, and 721.2.4
Columns	Thickness	12 inches	Tables 721.2.2, 721.2.3, and 721.2.4
	Cover	3 inches	Tables 721.2.2, 721.2.3, and 721.2.4

[1] For Carbonate – Aggregates and Restrained Construction

To achieve 1-hour fire rating, the minimum requirements are:

Table 20B: Summary of Minimum Thickness and Cover Requirements for One-Hour Fire Rating Construction

Structural Element	Minimum Requirements		IBC 2003 Reference
Slabs	Thickness	3.2 inches	Tables 721.2.2, 721.2.3, and 721.2.4
	Cover	3/4 inches	Tables 721.2.2, 721.2.3, and 721.2.4
Beams and Girders	Thickness	8 inches	Tables 721.2.2, 721.2.3, and 721.2.4
	Cover	1-1/2 inches	Tables 721.2.2, 721.2.3, and 721.2.4
Columns	Thickness	8 inches	Tables 721.2.2, 721.2.3, and 721.2.4
	Cover	1 inch	Tables 721.2.2, 721.2.3, and 721.2.4

[1] For Carbonate –Aggregates and Restrained Construction

Our testing determined that the average cover over the slab reinforcement is 1 inch in positive moment regions at the bottom of the slab, which meets or exceeds the average minimum cover requirements for fire rating. The concrete cover in both beams and girders varies between 2 inches and 2-1/2 inches and therefore exceeds the IBC minimum concrete cover requirements for a three-hour rating. The average concrete cover at columns is 0.9 inches and is less than that required by IBC for a three-hour, two-hour, or one-hour fire rating.

The choice, therefore is for the structural frame of SSTC to be modified to meet the requirements of Table 601 in IBC 2003 for a Type IA building type or, as we have been advised by Montgomery County DPS, the Owner could apply for another Code variance for the building to change its classification to a Type IIA building, in which case only one-hour ratings would be required for slabs, beams, girders, and columns.

XIV. OUR REVIEW OF SGH ASSESSMENTS AND PB RESPONSES

A. SGH Assessment

In their condition assessment, SGH's scope is noted solely to determine "…*deficient areas of one-way concrete slab [that] require remedial action and, if so, assist in the preparation of a detailed plan*…" with no notice as to materials testing. SGH treated SSTC as an existing structure and determined that a silane sealer would provide adequate serviceability.

Montgomery County DPS advised the SSTC facility cannot be considered to be "existing" as the building is still under construction and as an Occupancy Permit has not been issued. The following summarizes our comments related to the SGH assessments.

1. SGH's unique approach is to consider the SSTC is an existing structure (though Montgomery County DPS has advised it is not, since no occupancy permit has been issued) and therefore SGH utilizes ACI 318 Chapter 20 to perform their analysis. Note: Chapter 20, Section 20.1.1 indicates,

   "…*if there is doubt that a part of all of a structure meets the safety required of the Code, a strength evaluation should be carried out*."

   We agree with the determination as the structure is not complete and has not been subject to the full use loads from buses, etc. and therefore, Chapter 20 cannot be used for analysis.

   SGH's approach allowed it to utilize in their evaluation:

   a. Increased phi (Φ)factors
   b. Reduced load factors
   c. Concrete compressive strengths greater than 8,000 psi (e.g., 11,000 psi) based on cylinder tests, not in situ core strengths.

   SGH discusses the impact of its assumptions in its analysis and identifies the higher concrete strength based on the reported cylinder strengths (not in-place cores), and tension stress limits $12\sqrt{f'c}$ as important factors that influence the analysis of the structure they provided.

   However, SGH does not address the fact that because the structure is still under construction and is not occupied, the contractor is responsible for installing slabs that comply with Construction Documents, which, among other things, require slabs that are the correct thickness. Although we do not agree with SGH about each of their analytical

assumptions, we agree that the use of lower load factors is in accordance with the Project requirements, provided that they use matching phi factors. SGH's use of higher concrete compressive strength, especially in light of our materials testing results, is not appropriate.

2. SGH, using RAM© (not ADAPT©) limited service level stresses to 12√f'c. This limit assumes the structure is "cracked," which is not what the PB design Contract Documents intended. The use of tensile stress limits that are greater than 6√f'c is not appropriate since the WMATA Standards require, and PB identifies, 6√f'c as the limit that was used for the structure's design under service loads.

   Cracking is considered to initiate when the stress in the extreme tension fiber of a post-tensioned element exceeds 6√f'c. As the stress increases, the cracks propagate until the section is "fully cracked" (i.e., flexural cracks can no longer propagate due to the compression zone in the element). ACI considers this to occur when the stress in the extreme tension fiber is greater than or equal to 12√F'c.

3. SGH determined that a periodic and repeated application of a silane sealer would provide adequate serviceability. We do not agree with this recommendation because:

   a. The existing slab cracks will allow moisture to enter the slab in spite of the presence of a sealer and freeze and thaw exacerbating the cracks (it does not flow into cracks after installation)
   b. The sealer does not address the exposed slab post-tensioning tendons
   c. The sealer does not address tendons and steel reinforcement with concrete cover less than Code requirements.
   d. The sealer does not address physical damage that may occur during normal service.
   e. The sealer does not span "new cracks."

B. PB Response to SGH Assessment

PB's initial evaluation of the SGH report(s) determined that Level 330 slab stresses exceed the 6√f'c limit and that additional reinforcement was required. They also determined that additional reinforcement was required at the Level 350 slabs. Note that this applied to both "thick" and "thin" slabs. Based on these initial results, PB recommended that a bonded overlay be installed in the Level 330 and 350 drive aisles.

In follow-up calculations and correspondence, PB analysis results did not indicate that additional reinforcing was required but maintained that a bonded overlay should be installed.

A bonded overlay would provide adequate protection for the exposed tendons, and near surface tendons, and steel reinforcement. It would also allow some of the required strengthening to be installed and protected. The bonded overlay does, however, have several challenges:

1. Surface preparation is likely to damage near surface tendons and reinforcement.
2. Detailing of the overlay related to the existing cracks (even after epoxy injection) must be carefully done.

3. Changes in the existing driveway slab elevation would require changes to curbs, doors, railings, walkways, escalators, stairs, elevators, entrances, etc.
4. Owner/user expectations related to overlay performance, particularly as it relates to random cracking and isolated delaminations/spall.
5. Addressing existing structural slab restraint conditions that could lead to additional cracking.

XV. ENGINEERING ASSESSMENT OF COLLECTED INFORMATION TESTING AND ANALYSIS

A. Observations, Testing, and Structural Analysis

1. Concrete Strength

ACI 318-02 requires for analysis of in situ concrete that, of three cores taken in a pour, none can be less than 75% f'c and the average has to be greater than 85%. (WMATA only indicates the 85% average requirement as acceptable criteria.)

Based on ACI 318-02 core strength requirements, five slab pours do not meet the acceptance criteria of specified concrete strength. On Level 330, the pours with unacceptable concrete strength are 1A, 1B, 1E, and 1H, and on Level 350, pour 2C.

Pour Strips at Level 330 are unacceptable due to in situ reinforcing considerations and cracking.

Since the individual sets of three and cumulative core results yielded areas where those two conditions were not met, we also analyzed the concrete per ACI 214.4R-10 as described earlier herein.

Using ACI-provided analysis methods (ACI 214.4R-10 using 10 percent fractile and 90 percent confidence level), we estimate the slab compressive strength to be 6,970 psi.

2. Slabs

a. Typical Slabs

Our survey results indicate that the slab thickness varied from approximately 7 to 12-1/4 inches. The slab thickness variations (particularly in the "thin" slab areas) impact initial and service level stresses as well as shear and moment capacities.

The GPR data indicates that numerous tendons and reinforcing bars do not have the minimum specified concrete cover. Cracking and thin cementitious coatings were observed in the elevated slabs. The near-surface tendons and reinforcing bars as well as the observed cracking reduce the durability and/or fire rating of the slab.

The slab cracking is indicative of early age volumetric changes and restraint. Based on our review of the PB calculations and Contract Documents, there do

not appear to be provisions to mitigate cracking as a result of the as-designed restraint present in this structure. In addition, based on our analyses of the as-designed and as-built slabs, extreme fiber stresses exceeded the allowable initial and service level stress limits. In addition, Industry Standard limits on load balancing were also exceeded.

Our strength analysis of the as-built structure indicates that the in situ 10-3/8-inch and up to 12-1/4-inch thick post-tensioned slabs have adequate shear and moment capacity assuming 8,000 psi and 6,970 psi concrete. The analysis also indicates that the slabs have adequate moment capacity assuming a lower bound thickness of 8-1/2-8-3/4 inches for 8,000 psi concrete and 9 inches for 6,970 psi. Analysis of the slabs is influenced by several factors, including, but not limited to, slab thickness, concrete compressive strength, proximity to a support (beam), and assumed strip width for one-way slab behavior. Based on the results of this analysis, isolated shear strengthening of the slabs is required. Note that any necessary slab strengthening could be incorporated into the design of the slab overlay, when installed.

b. <u>Level 330 Pour Strips</u>

The Pour Strip slabs are not post-tensioned and the West Pour Strip is missing North-South temperature and shrinkage reinforcement. As such, they were not constructed in accordance with the Contract Documents. Analysis results indicate that these slabs do not have sufficient shear or moment capacity to resist the design loads with concrete strength of 8,000 psi or 6,970 psi by inspection.

Due to the observed cracking, thin cementitious patches in places, and lack of adequate strength, the slabs should be removed and replaced.

3. Beams

Cracking and relatively minor (except for two beams) isolated tendon deviations were observed in the beams. Our as-designed and as-built analyses indicate that the initial and service level extreme fiber stresses exceed the required limits. The location and orientation of the observed cracking, combined with the elevated stress levels, indicate that most of the observed cracking at the ends of the beams is due to the application of as-designed post-tensioning.

Strength analysis results indicate that the beams generally have sufficient flexural capacity using 8,000 psi concrete but beams spanning more than approximately 28 feet that are located below the drive aisles are under strength in shear when using 5,000, 6,970 or 8,000 psi concrete.

Combined shear and torsion analysis results indicate that the floor beams do not comply with ACI 318-02 requirements for closed stirrups since the design torsion load exceeds the beam’s torsion threshold. The perimeter spandrels (under 8,000 and 6,970 psi strengths) also lack adequate strength to resist the shear and torsion loads.

4. Girders

Cracking and relatively minor and isolated as-placed tendon deviations were observed in the girders. Our as-designed and as-built analyses indicate that the initial and service level extreme fiber stresses exceeded the Code required limits.

Strength analysis results indicate that the girders have sufficient flexural and shear capacity using 8,000 and 6,970 psi concrete. However, combined shear and torsion analysis results indicate that strengthening is required.

5. Columns

Cracking and near-surface reinforcing bars were observed at the columns. The cracking appears to be related to girder post-tensioning forces induced into the columns as well as reflective cracking at near-surface bars. The near-surface column bars, which are not epoxy coated, affect durability and fire resistivity. In order to protect the near-surface reinforcement and regain the required the fire resistivity, the columns should be encased, preferably in concrete. The columns have adequate strength to resist the design loads.

B. Durability Analysis

1. Slabs

Widespread cracking was visually observed on the top and bottom surfaces of the structural slabs at both Level 330 and Level 350. In general, cracks at the top surface of the slab have the most significant effect on the long-term durability, as moisture and chloride-ions are able to collect and migrate into the concrete at these cracks. Though cracks were noted on the faces of beams and girders and on the soffit of the slabs, they are not considered to be as critical to the long-term durability of the structure since these cracks do not experience as severe an exposure to moisture and de-icing salts as cracks at the top surface of the slab.

Representative cracks were measured to be between 1/200th inch and 1/64th inch wide at the top surface of the slabs. Non-destructive evaluation of representative cracks indicates that the crack depths vary from 1/2 inch to the full thickness of the slab. The average depth of cracks evaluated extend to the mid-depth of the slab, and based on the standard deviation of measured crack depths, the majority of cracks extend between 2 inches and 7 inches from the top surface of the slab. As these depths are greater than the depth of the top reinforcement, moisture and chloride ions are considered to have direct access to the reinforcement, which allows corrosion at these locations to initiate immediately.

In addition, cracks at the top surface of the slab will retain moisture in the crack during freeze-thaw cycles. Repeated expansion of water as it freezes in the crack can cause additional premature distress to develop in the concrete proximate to the cracks, and

will cause the crack widths, lengths, and depths to extend over repeated cycles of freeze-thaw.

Post-tensioning ducts that were observed to be exposed at the top surface of the post-tensioned slabs or near surface on Levels 330 and 350 are susceptible to premature deterioration in their current condition. Despite the tendons being encased in grout and encapsulated in ducts, the ducts are vulnerable to abrading and splitting when exposed to vehicular traffic, which will only be exacerbated when vehicles are equipped with chains during the winter months. Once this occurs, chloride ions will be able to penetrate through the grout over time to the post-tensioned tendons.

2. Pour Strips

Non-destructive evaluation of the Pour Strips revealed that the Pour Strip at Level 330 was constructed with mild steel reinforcement spacing that does not comply with Contract Documents. The Pour Strip between Column Lines 10 and 11 at Level 330 was found with temperature reinforcement spaced at 51 inches on center, while the Contract Documents require temperature reinforcement at 12 inches on center. Insufficient thermal reinforcement can allow cracks to develop and to extend, which in turn can allow moisture and chloride ions to infiltrate into the Pour Strip.

Pour Strips are particularly sensitive locations with respect to the long-term durability of post-tensioned structures since post-tensioning anchors are located along the edges of the Pour Strip for adjacent pours. There is no caulk in these joints, just a simple vee joint. Chlorides that are able to infiltrate to these anchors through cracks in the Pour Strip can initiate corrosion in these anchors, if the encapsulation is compromised. If the anchor caps have been displaced, corrosion of post-tensioning anchors can result in loss of stress in the post-tensioning tendons over time, which will adversely affect the long-term structural capacity of the Pour Strips. Caulking installed in these joints would limit that possibility.

3. Columns

Cracked concrete columns are also susceptible to premature deterioration due to exposure to moisture and chloride ions from de-icing salts. Unlike the soffit of the slab, beams, or girders, the bases of concrete columns are close enough to the vehicle lanes to be considered as splash zones where passing trucks will repeatedly wet the surface with moisture containing deicing salts. Since low concrete covers and no epoxy coating (by design) were measured on the columns, the time to initiate the corrosion in the reinforcing steel is reduced. The average minimum depth of concrete cover was 0.9 inches, and may even be as low as 0.3 inches at a single point on a column. Based on the service life model, corrosion can initiate at 20 years with one inch of cover. In addition, cracks observed on the columns will allow corrosion to start immediately at those locations.

4. Freeze-Thaw Resistance

The resistance of a concrete from deterioration due to repeated freeze and thaw cycles is provided by entrained air (via concrete additives) in the concrete material, and by air entrapped during placement and mixing, though to a much lesser degree than entrained air. Air entrainment is achieved in concrete through the introduction of admixtures, which increase surface tension in the plastic concrete and result in small diameter (typically 10 microns), nominally uniformly distributed air bubbles in the concrete.

Entrained air prevents freeze-thaw distress from occurring by allowing pore water in the hardened concrete to have locations to expand in as it freezes. The adequacy of air entrainment is related to both the amount of entrained air and the spacing between air voids in the hardened concrete. Air contents at many in situ tested locations in the as-built concrete do not meet the requirements of the Contract Documents, making the concrete vulnerable to freeze-thaw damage over its full design life. The air contents measured by RBB at times do not conform with the Contract Document requirements.

These values do not conform to the Contract Document requirements, which are conflicting, but the most stringent calls for air content from 5.5% ± 1/2%.

Core samples indicated air void spacing factors (which is not a Contract Document limitation) in excess of 0.008 inches, but published literature indicates that spacing factors greater than 0.008 inches may experience reduced freeze-thaw performance.

5. Service Life Modeling

Service life modeling was performed using STADIUM© to evaluate the time until corrosion can initiate in the reinforcement. This analysis considers the chemical composition of the concrete, the initial chloride content of the concrete, and the concrete transport properties. This statement refers to the fact that the service life model does not calculate times to corrosion initiation that account for the presence of cracks or other defects in the concrete.

Estimates of service life are only valid at locations where concrete is uncracked. As discussed in the report, at locations where the concrete is cracked, chlorides are able to access the mild reinforcement immediately, and therefore corrosion is considered to initiate immediately at these locations. The transport properties used in the STADIUM© model are based on testing performed on concrete samples excised from the structure, thus they represent the “as-placed” concrete properties. Based on this information, the Service Life analysis estimates the time for chlorides to penetrate the concrete to the depth of the reinforcing steel.

Chloride penetration analysis estimates time to corrosion based on properties of the concrete and depth of cover to reinforcing steel. Based on information taken from cores, embedded reinforcing steel is likely to begin corroding between 18 years for a low diffusion coefficient to greater than 50 years for concrete with a moderate to high diffusion coefficient, as previously described, when the cover thickness is 1 inch. However, locations with no cover, such as at locations with near surface reinforcing

steel or where post-tensioning ducts are exposed to the surface, will begin to corrode immediately and once the duct is compromised for the product.

The durability of a concrete structure is reduced as the depth of concrete cover over reinforcement is decreased. This relationship is a result of the fact that there is a smaller distance through which chlorides must penetrate to reach the depth of the reinforcing steel to initiate corrosion. At locations with cracks, there is a direct path to the reinforcing steel, and corrosion initiates immediately.

Epoxy coating is intended to protect the steel reinforcing bars in slabs, beams, and girders. At locations where the epoxy coating on the bars is intact, corrosion cannot initiate. However, it is known that all epoxy bars have holidays (discontinuities or nicks) or the coating might get damaged during construction, and in fact, the Contract Document specifications allow up to "*2% of damaged coating in each twelve inch bar length*." At these locations, the steel will corrode, and since the corrosion is limited to a small area, the rate of corrosion at that location is very high. Based on this, the Service Life model indicates a lower bound time to initiate corrosion.

DCI Corrosion inhibiting admixture was specified to be added with no quantity noted, sampling "*where indicated*" by the Contract Documents, yet none were included. The approved mix designs (mix #8K2DC2NL) placed in the slabs that included corrosion inhibitor as a part of the mix design. Two gallons per cubic yard were specified in that approved mix design.

By adding DCI to the concrete, the threshold to initiate corrosion is increased, and thus the time to initiate corrosion is also increased. Nonetheless, DCI does not prevent corrosion from initiating at locations where cover is known to be zero; corrosion at these locations will initiate immediately in the reinforcing steel and post-tensioning once the post-tensioned duct and grout are compromised.

In short, the durability of the in situ concrete decks of SSTC do not meet the 50-year useful life criteria as per WMATA requirements.

XVI. CONCEPTUAL REMEDIATION RECOMMENDATIONS

Our condition assessment results indicate that remedial work is required at the slabs, columns, beams, and girders. Other nonstructural work will be required to deal with the access issues created by the remediation method chosen.

Based on our analysis of the structure, the following conceptual recommendations are made.

A. Combined shear and torsion strengthening is required at selected beams and girders.

    1. Shear strengthening of the beams could be accomplished using fiber reinforced polymer (FRP) sheets or rods. Alternatively, a bonded overlay may provide sufficient additional shear capacity. Addressing the torsion loads is more challenging. The optimal solution would be to close and possibly supplement the existing stirrups or install a beam padout

adjacent to the sides of the beams tied to the beams with drilled-in dowels (ties). Obviously, any strengthening solution must be protected from fire.

2. Shear strengthening of the girders could be accomplished using FRP sheets or rods. Alternatively, a bonded overlay may provide sufficient additional shear capacity. Addressing the torsion loads is more challenging. The optimal solution would be to close and/or possibly supplement the existing stirrups or install a girder pad out adjacent to the girders tied to the girders with drilled-in dowels (ties). Obviously, any strengthening solution must be protected from fire.

B. Enlargement of the columns with insufficient concrete cover between Levels 330 and 350 to provide the required fire resistivity. This will also increase durability.

C. Shear strengthening will be required for limited slab areas that are less than 8-3/4 inches thick with 8,000 psi concrete or 9 inches thick with 6,970 psi concrete. Note that the areas that require strengthening are located immediately adjacent to beams.

D. Existing Pour Strip slabs on Level 330 must be replaced with appropriately designed and detailed Pour Strips

E. A concrete overlay solution to the top surface of the Level 330 and 350 slabs should be provided for the SSTC concrete framed decks of both Levels 330 and 350 to address near-surface post-tensioning tendons and reinforcement. Note that any required slab shear strengthening could be incorporated into the overlay design.

   There are two approaches that can be adopted to accomplish this:

   1. Design a plaza waterproofing system with an appropriately designed wearing course for heavy transit traffic loads. There may be a need to strengthen the structure to accommodate the additional dead loads being applied to the structure beyond the 35 psf that is included in the original design;

   or

   2. Design a bonded topping slab properly detailed to minimized thermal and restraint force cracking. Again, there may be a need to strengthen the structure to accommodate the additional dead loads being applied to the structure beyond the 35 psf that is included in the original design.

   In the overlay solution, railings will have to be added at pedestrian access locations from the drive aisles (curb height less than Code-required) and ramps cut into the walkways to provide pedestrian access “lanes.” Railings will have to be located to allow for bus egress and boarding.

   Please note that attention needs to be given to normal and expected ongoing maintenance required for exposed structures of this type over the life of the structure. This means that to achieve the intended Service Life, some routine, periodic maintenance will be required, such as maintaining expansion joints, injecting cracks, periodic wash downs, etc.

The "Owner" should apply for a Building Code variance to allow the building to be classified as a Type IIA building.

XVII. CONCLUSIONS

The in situ conditions at SSTC have been caused in varying degrees by errors and omissions of the designer, PB (Parsons Brinckerhoff), the contractor, FP (Foulger-Pratt Contracting, LLC) and its subcontractors, and the inspection and materials testing firm and Special Inspections Program Special Inspector, RBB (The Robert B. Balter Company).

Each of those contributions is noted throughout this report. Our conclusions are summarized as follows:

A. Based on our review of the information provided, the design depicted in/on the Contract Documents was not prepared in accordance with the applicable Building Code(s), the WMATA Manual of Design Criteria or Industry Standards. Based on our analysis, failure of the design to follow applicable codes and standards resulted in widespread cracking in the slabs, beams, and girders, and reductions of minimum concrete cover requirements.

B. Based on our review of the information provided, the independent inspectors, Special Inspections Program Special Inspector, Quality Assurance, Quality Control, etc., did not raise sufficient concern regarding the numerous issues that were known and/or became visible in the concrete during construction, apparently did not follow up on solutions to those issues, and did not perform their services in accordance with Industry Standard, their Contract, or the Statement of Special Inspections.

C. Based on our review of the information provided, the Contractor did not construct structural elements of the SSTC facility in accordance with the Contract Documents, ASIs, and RFI responses. The Contractor, among other things as detailed herein, placed concrete materials not in accordance with the Contract Documents.

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We do note, as stated in the Executive Summary, in our professional opinion and with reasonable degree of engineering certainty, the building can safely support the current construction-phase loading and, with the conceptual remediations completed as outlined herein after remediation documents are prepared, can safely carry the full Code and WMATA required loads.

Very truly yours,

Allyn E. Kilsheimer, PE
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KCE Structural Engineers, PC
MD Reg. # 6774
Valid until 12/30/2013

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ALLYN ERIC KILSHEIMER
No. 6774
REGISTERED
PROFESSIONAL ENGINEER

Professional Certification. I hereby certify that these documents were prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the State of Maryland.
License No. 6774, Expiration Date: 12-30-2013.

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